Proceedings of the 2016 Coal Operators' Conference

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SPONSORS AND EXHIBITORS
On behalf of the conference editorial and advisory board and I welcome you to the 16th Coal Operators' Conference (Coal2016). Since its beginning in 1998 it has seen authors presenting papers on a wide variety of topics. The availability of past papers on line 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.

The year 2015 can be appropriately described as “annus horribilis” for the Australian and worldwide coal mining industry. It is the year marked with the drop in coal price, a record number of mine closures and redundancies as well as the signing of the Paris Agreement on climate change. It is also the year of improved production capacities of the modern mines from both underground and surface operations. The Coal Operators’ Conference has played a significant role in bringing about and making the industry aware of technological improvement and modernisation that is taking place in the coal mining industry. Many new ideas, innovations and safety improvements have been the centre of discussion and documentation in this conference. At present there are 586 published papers on line and documented in 15 proceedings. These papers have attracted more than 561000 as of January, 2016, since going online in 2008. An additional 51 papers will be added on line from this conference. These papers are mostly from Australia but include papers from Canada, China, Czech Republic, Germany, India, Iran, Kazakhstan, New Zealand, Turkey, UK, USA, and others.

The publicity provided by International Coal News (ICN) is deeply appreciated and thanks to Lou Caruana for maintaining support for our conference.

Special thanks to the editorial panel members, paper reviewers and others support. Thanks to Ali Mirzaghorbanali for typesetting the conference proceedings; Samand Asi and Shahin Aziz, for the conference website development and general management of the conference, Kevin Marston and Ray Tolhurst for their continuing logistics support and the control of the conference finances. Congratulations to Ray for his joint awards; The Australian Day City of Wollongong Senior Citizen of the year award, 2016 and the Aus IMM Beryl Jacka award.

Special thanks to the University of Wollongong printery staff Terry Campani for designing the conference proceedings cover page, Garry Piggott and Maria O’Hearn for printing the conference proceedings, and Uni-Centre for catering. Finally sincere thanks to authors and participants, who form the backbone of the conference success.

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

Professor Naj Aziz
Conference chairman and convenor
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FURTHER CONSIDERATIONS ON THE APPLICATION OF DE-BONDED CABLES

Ross Seedsman

ABSTRACT: De-bonding of the central portion of long tendons has been suggested if they are installed in low strength rock prior to a significant stress change, be that either at the maingate corner or in installation roadways. The purpose of the de-bonded section is to allow a reduction of the strains applied to the cables during failure of the rock mass and so increase the chance of them surviving large shear movements. A recent application of de-bonded cables in a longwall installation roadway resulted in a major reduction in roof deformations from in excess of 100 mm to less than 3 mm. Assuming similar rock strengths and ground stresses, the de-bonded cables apparently prevented the onset of vertical dilation in the roof. Compared to an installed shear resistance of 183 kPa, shear stresses of up to 1.5 MPa are predicted in the roof immediately above the bolted horizon. A mechanism is proposed that involves the onset of brittle failure followed immediately by a large amount of sub-horizontal shear displacement. Because of the free length of the de-bonded cables the induced strains were insufficient to cause the cable to rupture. The de-bonded cable was post-tensioned and this led to the closure of some of the dilated bedding induced on the first pass.

BACKGROUND

Seedsman (2015) proposed the use of de-bonded cables to improve the survivability of cables during stress-induced boundary crushing so that they would be available for later suspension of the failed material. That paper included a case study of a longwall installation road where roof movements at the 2.5 m horizon during the second pass exceeded 100 mm while movements at the 5.5 m horizon were less than 10 mm. In the last 12 months the same mine developed the next installation road on the other side of a 50 m wide chain pillar. De-bonded cables were used and the movements at all levels during the second pass were mostly zero, and never more than 3 mm. Whilst reduced movements were anticipated, the lack of any movement was not.

The performance of the latest installation roadway could be explained by:

1. Different geology, although the site investigation did not reveal major differences in lithologies or strengths.
2. Different stress field, possibly indicated by the poorer conditions in the stress relieving roadway. There was no intervening fault or other geological anomaly that could provide a change in the stress orientation.
4. This paper speculates on the behaviour of the de-bonded cables.

LOADING OF CABLES IN THE LABORATORY

Tension tests

Typical elongation properties of the wire used in long tendons are 0.85 % of the length at yield and 5.6 % at failure. With this information it is possible to calculate the deformation properties of the free length of a cable of any length under simple direct tensile loading (Figure 1). For a Double Embedment Test (DEPT), Clifford et al (2001) provided data on the deformation of a fully grouted double birdcage cable (Figure 1) which showed that the cable broke at a joint opening of about 9 mm. This result implies a free length of approximately 70 mm – however a direct comparison is not valid as the DEPT test result

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includes the impact of the bulbing of the cable. Other DEPT data for a typical Australian plain strand cable indicates a displacement of about 18 mm – hence an implied free length of 320 mm. It is noted that the elastic and yield deformations of the steel cannot be readily identified in the laboratory tests as they are masked to some degree by the yield and failure at the cable/grout interface which, in the Clifford test data, may be indicated at about 1 mm/300 kN.

Figure 1: Calculations of the deformation of cables based on supplied mechanical parameters and results of a double embedment pull test (after Clifford et al., 2001)

Shear tests

During 2015 a number of laboratory studies of the performance of cable bolts in shear (Figures 2 and 3) were published (Aziz et al, 2015, McKenzie and King 2015). Key results have been:

- High stiffness for the first 5 mm -10 mm of shear movement – probably the limit of the elastic behaviour of the grout ahead of the tendon (ACARP, 1995).
- Fully grouted cables breaking in tension at a shear displacement of about 50 mm (McKenzie and King 2015) or 60 mm to 80 mm (Aziz et al., 2015) of shear movement.
- No failure of de-bonded cables before 120 mm of shear displacement.

Figure 2: Shear loads-displacement graphs and a photo showing crushing of the grout, bending of the cable wires and their dominantly tensile failure (McKenzie and King 2015)
It is of value to convert these laboratory shear strengths to an installed shear resistance for typical Australian support densities. For this case study the density was 3.3 cables per metre for a 9 m span giving an installed shear capacity of 183 kPa (3.3*500/9).

**BOND STRENGTH IN THE FIELD**

Assuming that there are no stress changes applied to the borehole wall after the cable is installed, the performance of the steel/grout or grout/rock interfaces is determined by the lower of the undrained shear strength (=cohesion) of either the grout or the rock. Increased tensile loads applied to the tendon will induce shear movement at the weaker of the two interfaces, recognising that the shear stresses will be greater at the steel/grout interface because of the lower surface area.

The shear strength of typical chemical resins, as determined by punch shear tests is approximately 30 MPa with a typical uniaxial compressive strength (UCS) of 56 MPa and a Youngs Modulus of 4 GPa (Aziz et al., 2014). For cement grouts, the UCS is a function of the water:cement ratio and at a ratio of 35-40% the UCS is typically in the order of 60 MPa with a Youngs Modulus of about 12 GPa (Hutchinson and Diederichs, 1996): a cohesion of 15.3 MPa at a water:cement ratio of 40 % is quoted by Moosavi and Bawden (2003). For typical friction angles, the cohesion of coal measure rocks is about 30% of the UCS so at 50 MPa the cohesion is 15 MPa.

Based on the method of Littlejohn (1993) for stronger rocks failure will be in the grout or the resin, but for weaker rocks encapsulated with resin failure will be in the rock (Figure 4). It is noteworthy that the analyses in Hutchinson and Diederichs (1996) are particularly focussed on the failure in cement grouts.

**Figure 4: Typical calculations of anchorage capacity with no post-installation stress change**
Anchorage capacity changes if there is a stress change in the rock mass after the cable is installed. For a stress reduction, the borehole expands and if this expansion exceeds the roughness of the borehole wall there will be a loss of anchorage – this is also the reason cables are bulbed and why they should always be installed with a plate. For a stress increase, there will be mobilisation of an additional frictional strength that increases the efficiency of the anchorage.

POSSIBLE LOADING OF CABLES DURING THE SECOND PASS OF AN INSTALLATION ROADWAY

The same Transverse Isotropic Brittle (TIB) elastic model as used previously (Seedsman 2015) has been re-examined using Examine2D. In this model the UCS is 30 MPa, Youngs Modulus is 7.5 GPa and the Independent Shear Modulus is 500 MPa. The initial stress regime involves a horizontal major principal stress of 11.25 MPa and a vertical stress of 6.25 MPa. In Figure 5, the grey shaded area is where the rock has failed during the second pass mining according to the TIB model. The black zones immediately above the second pass driveage indicate the onset of tension related to relaxation into the softened zone above the first pass. A cross-plot of the final elastic stresses at each point in the prediction grid indicates that failure was primarily due to a reduction in the minor (vertical) stress and not the result of a concentration of the horizontal stresses.

A small elastic stress reduction in a horizontal plane is indicated prior to the onset of TIB failure and once failure is initiated the stresses normal to the borehole will reduce to zero. Above the TIB failed zone, at heights greater than 9.5 m, there is an increase in the horizontal stresses, and there is no change at the bolted horizon as this was de-stressed on driveage and hence before cables 1 – 3 were installed.

The expansion of a 42 mm diameter borehole in 7.5 GPa modulus rock when the confining stress is removed is estimated to be 0.04 mm. This expansion is assessed to be negligible compared to the likely roughness of the borehole wall and hence the bond strength for fully grouted cables is unlikely to have been adversely affected. For de-bonded cables, the stress reduction within the TIB failed zone is not a factor.

The continuous contour lines in Figure 5 are the shear stresses that developed during the second pass and hence after cables 1-3 were installed. Prior to the onset of TIB failure, the shear stresses imposed on cables 1 – 3 are in excess of 500 kPa and can exceed 1.5 MPa within 3.5 m of the roof line. Clearly these shear stresses are in excess of the installed capacity of the cables which is in the order of 180 kPa for 5 cables every 1.5 m across a 9 m span. Cables 4 and 5 were installed during the second pass and most of the shear stresses shown in Figure 5 would have already developed.

The dashed lines in Figure 5 are the horizontal movements that were obtained when the excavation was remodelled to include the grey TIB failure zone. They may be interpreted to be the maximum amount of shear movement that was imposed onto the TIB failure zone. At heights less than 4.5 m above the roof line, the total amount of closure movement is in the order of 50 mm to 60 mm. From the Examine2D output it is possible to calculate the orientation of the principal stress and from that the orientation of the likely shear surfaces that would be induced. Cable 2 is exposed to a horizontal shear surface and cable 3 exposed to a shear surface dipping at 20°. At cable 3, 60 mm of horizontal movement would produce 64 mm of movement along such a shear plane with 22 mm of vertical displacement.

The predicted movement is sensitive to the modulus/shear modulus ratio – a value of 20 instead of 15 increases the movements by 16 %.
DISCUSSION

In Figure 6 typical results for grouted and de-bonded cables based on McKenzie and King (2015) are plotted for the case of 5 cables installed every 1.5m across a 9 m wide roadway. The shear demand lines are taken from Figure 5 assuming there is a linear relationship between the elastic shear stress before failure and final displacements after failure (Table 1). Figure 6 is in the form of a ground reaction curve (Brady and Brown, 1985) for which the interpretation is that stability is achieved if the support line intersects the ground characteristic – in this case the shear characteristic. Inspection of the figure indicates that fully grouted cables would be ruptured 2.5 m above the roof line but not ruptured at 4.5 m above the roof line. De-bonded cables would not rupture at either location.

Table 1: Linear shear demand characteristic derived from the TIB failure model

<table>
<thead>
<tr>
<th>Height</th>
<th>Shear movement (mm)</th>
<th>Shear stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 m – 4.5 m above roof line</td>
<td>0</td>
<td>500</td>
</tr>
<tr>
<td>6 m – 2.5 m above roof line</td>
<td>0</td>
<td>1250</td>
</tr>
<tr>
<td>6 m – 2.5 m above roof line</td>
<td>0</td>
<td>1250</td>
</tr>
</tbody>
</table>

Figure 5: Simple elastic model using the TIB failure criterion, left - contours of shear stresses (in MPa) and horizontal displacements (in m), grey zone is TIB failure, black zone is tensile failure; right – final principal stress points. Cables 1-3 installed on the first pass, cables 4 and 5 installed on the second pass.
In terms of roof extensometry, the rupture of cables would result in the potential for unrestrained movement of the roof below. The actual movement cannot be predicted as it is possible that some sort of arching will develop in the bolted/meshed/possibly grouted immediate roof. For the de-bonded cable, a shear movement of 60 mm could produce up to 22 mm of vertical displacement at cable 3. However at the same time the cable will have been tensioned to approximately 30 tonnes. Given the cable density, 30 tonnes is equivalent to the dead weight load of 6.6 m of rock (6.6*1.8*2.5). It is therefore suggested that the result of the shearing of a de-bonded cable is the post tensioning of the cable and the possible closure of any dilated bedding surfaces below (Figure 7).
CONCLUSIONS

The much lower roof deformations recorded in the case study using de-bonded cables may be related to the cables being post-tensioned but not ruptured during the onset of failure in the rock. The way in which this develops can be considered in a number of steps:

1. Cables 1 – 3 are installed in the first pass, prior to the second pass. There is a thin zone of failure in the immediate roof, the consequences of which are controlled by the primary bolts and mesh. The installed shear capacity of the cables is about 180 kPa. There is some minor bedding dilation in the roof before the extensometers are installed.

2. Stripping of the roadway during the second pass results in a large reduction in the minor stress in the roof above the first pass and an associated small reduction in the major stress. There are no material changes to the bonding capacity of either grouted or de-bonded cables. Shear stresses of up to 1.5 MPa are induced.

3. Brittle failure in the rock develops and cannot be prevented by the cables and the shear stresses are reduced to zero.

4. Low angle shear surfaces are created, coincident with the horizontal closure of the intact rock either side of the brittle failure zone.

5. At about 2.5 m into the roof, the shear displacements are sufficient to rupture fully grouted cables: de-bonded cables have lower shear stiffness and survive.

6. Higher in the roof the displacements are less and the fully grouted cables are not ruptured.

7. For grouted cables, the roof below 2.5 m is now unrestrained and the prevention of collapse requires arching within a bolted/meshed rock mass. Higher in the roof the grouted cables are not ruptured and suspend the rest of the brittle failure zone.

8. De-bonded cables do not rupture, and all of the failed rock is suspended by the cables. The grouting at the collars ensures the full load capacity of the cables can be mobilised.

9. Shear loading of the de-bonded cables effectively post-tensions the cables and any dilation in the immediate roof may be closed.

REFERENCES


Seedsman, R W, 2013. Practical strength criterion for coal mine roof support design in laminated soft rocks, Mining Technology, 122(4), 243-249.

MODELLING SIMULATION OF THE PERFORMANCE OF CABLE BOLTS IN SHEAR

Haleh Rasekh, Ali Mirza, Naj Aziz, Jan Nemcik and Xuwei Li

ABSTRACT: The application of cable bolts as a secondary support system in coal mines has substantially increased over the last decades. This development emphasises a prerequisite for a better understanding of the mechanical behaviour of cable bolts in combination of shear and axial loads, which is of interest to geotechnical engineers and designers of underground structures. Based on various research studies undertaken, analytical models have been proposed to assess the shear performance of cable bolts with the aim of designing safe and sustainable rock structures. Recently, a shear strength model for cable bolts was proposed by Aziz et al. (2015). The model was based on Mohr-Coulomb failure criterion and the Fourier series concept. The proposed analytical model was associated with a set of systematic experimental studies by which model coefficients were calibrated. This paper compares the performance of a number of proposed mathematical models against the experimental data and presents a model that simulates the shear behaviour of cable bolts, which is in agreement with the experimental results.

INTRODUCTION

Since the 1960s, the use of cable bolts as long fully grouted elements in the mining industry was started with the advantageous for rock mass to support itself (Fuller and O'Grady 1993). Cable bolts are flexible tendons with high tensile strength that contain a group of steel wires, which are twisted into strands. It provides reinforcement and support in mining and civil engineering excavations. Cable bolts are applicable in various areas of walls, roof and floor of underground and surface openings, including: drifts and intersections, they are also used for metalliferous mining open stope backs, open stope walls, cut and fill stopes, draw points and permanent openings (Hutchinson and Diederichs 1996; Windsor 1992; Fuller 1983 and Puhakka 1997).

Cable bolt profiles come in three types: plain, spiral and indented as shown in Figure 1. The performance of cable bolt changes, depending on profile.

![Cable bolts profile: plain, spiral and indented](image_url)

Figure 1: Cable bolts profile: plain, spiral and indented

Pull out tests and shear tests are different methodologies to evaluate the behaviour of cable bolts. Assessing the tensile failure and load transfer capacity of cable bolts by using the pull out test methodology was reported by Hawkes and Evans (1951), Fuller et al., (1978), Diederichs (1993), Bouteldja (2000) and Morsy and Han (2004). Nowadays, the performance of cable bolts under shearing...
is a topic of interest to many researchers. Cable bolt failure in a field is a combination of shear and tensile loads. Shear test can be conducted in the laboratory by using either the single shear or double shear tests. The single shear test as an approach to determine the shear strength of cable bolts has been conducted by a number of researchers while limited studies have been carried out using double shear tests. A number of mathematical models have been proposed to calculate the shear strength of fully grouted cable bolts. Six analytical models will be summarised in the following part and compared with measured test data at the UOW laboratory.

LITERATURE REVIEW

Different analytical models are available to determine the shear strength of fully grouted cable bolts. Details of six of these are set out below and it should be noted that most of these involved single shear testing. Only the tests by Aziz et al., (2015) involve double shear testing.

Dulacska (1972) conducted 15 single shear tests by using different concrete grades, three steel sizes and four different angles for stirrup to examine the action of dowel in cracked concrete to establish theoretical load-deformation relationships. The steel bars near the cracks and the friction across the concrete surfaces counteract the slipping of the cracks. The shear force was modelled by the following equation:

\[ T = 0.2d_b^2\sigma_y \sin \delta \left[ 1 + \frac{\sigma_c}{0.03\sigma_y \sin^2 \delta} - 1 \right] \]  

(1)

where, \( T \) is the shear force carried by the bolt, \( d_b \) is bolt diameter, \( \sigma_c \) is Uniaxial Compression Strength (UCS) of rock, \( \sigma_y \) is yield stress of bolt, and \( \delta \) is angle of stirrups.

Bjurstrom (1974) directed a series of single shear tests to find out the shear forces transfer in grouted untensioned bolted joints. The granite blocks with natural and artificial joints and different normal pressures were tested. The joint surface was smooth and the total shear displacement was less than 50 mm. 50 tests were carried out without bolt installation for verification of the friction and 60 tests were with bolt installation. The shear displacement was measured by differential transformer (LVDT). As shown in Figure 2, the shear strength was determined by three components: friction, tension of the bolt and the dowel effect. Also, the results demonstrated that when the angle between the bolt and joint is less than 35°, the failure occurs in tension. By increasing this angle to 40-45°, the failure will be a combination of shear and tension.

Figure 2: Relationships between forces caused by friction, tension of bolt and dowel effect and shear displacement for a joint reinforced by grouted untensioned bolts (Bjurstrom 1974)
The shear strength is calculated by following equations:

- **Reinforcement effect** \( T_b = p(\cos \beta + \sin \beta \tan \varphi) \) \hspace{1cm} (2)
- **Dowel effect** \( T_d = 0.67d_p^2 \sqrt{\sigma_c \sigma_y} \) \hspace{1cm} (3)
- **Friction of joint** \( T_f = A_j \sigma_n \tan \varphi \) \hspace{1cm} (4)

where, \( p \) is the axial load corresponding to shear displacement, \( \beta \) is the initial angle between bolt and the joint, \( \varphi \) is friction angle, \( d_p \) is bolt diameter, \( \sigma_y \) is yield strength of bolt, \( \sigma_c \) is UCS of the rock, \( A_j \) is joint area, \( \sigma_n \) is normal stress on joint.

Haas (1976) conducted single shear tests on blocks of chalk and limestone. There were different variables in tests such as: type of bolt, normal pressure on interfaces and different orientations of bolts due to the shear surface (0°, +45° and -45°). The effect of using rock bolts was investigated by increasing the resistance of rock in shear displacement along the fractured surfaces. Results indicated that increasing the shear resistance is the result of applying 170 kPa of normal compressive stress on the plane and also orienting the bolt in the direction of shear progress. However, the single shear test is due to non-equilibrium load distribution along the joint plane (Figure 3).

![Figure 3: Non-equilibrium in vicinity of the shear joint (Haas 1976)](image)

Haas (1976) also proposed a mathematical model for calculating the shear resistance of bolted rock. The average shear stress is determined by:

\[
\tau_{ave} = \tau_0 + \frac{\mu T_0 \cos \theta + T_0 \sin \theta}{A_s}
\] \hspace{1cm} (5)

where, \( \tau_{ave} \) is the average shear stress with bolt for impending movement, \( \tau_0 \) is the average shear stress on the interface without a bolt, \( \mu \) is coefficient of friction, \( \theta \) is initial orientation of bolt, \( A_s \) is area in shear, and \( T_0 \) is tension in the installed bolt.

Spang and Egger (1990) studied the shear performance of a fully grouted untensioned bolts in jointed rock by single shear test methodology. They used numerical studies by Finite Element Method for a three dimensional model. Bolt inclination to the joint surface was considered at 30° and 90°. Three stages of behaviour for a fully grouted bolt in the jointed rock mass were defined as elastic, yield and plastic stages. The behaviour in the elastic zone followed the Mohr-Coulomb relationship. It was monitored that the yield stage in the bolt and mortar occurs at the very first stages of loading when the shear displacement was 1 mm and the bolt only reached 10% of its ultimate strength. Therefore, the behaviour was mostly governed in the plastic zone. The Drucker-Prager and von Misses failure criteria were used to define the elastic-plastic behaviour of materials. The joint-friction of blocks was neglected for simplification in the model with blocks being held together by the grouted bolt.
Results from tests showed the higher shear resistance for an inclined bolt compared to the perpendicular one. Also, the deformation amount in the inclined bolt was less than the bolt without inclination. The failure in the perpendicular bolt was attributed to bending that combined shear and tension across the joint surface. On the other hand, the failure for the inclined bolt was due to the tension near the shear surface. As shown in Figure 4, the plastic strain in the inclined bolt (30°) is less than the perpendicular one.

Figure 4: Plastic strains in the bolt for T=30KN and $\alpha = 0$ and 30° (Spang and Egger 1990)

The gap created due to the shear force and failure of the bond between steel and mortar for the perpendicular bolt, was more than the inclined bolt (Figure 5).

Figure 5: Gap between bolt and mortar for $\alpha = 0^\circ$ (T = 25KN) and 30° (T = 25, 35KN) (Spang and Egger 1990)

The proposed equation to calculate the shear resistance of a bolted joint is:

$$T_0 = P_t \left[ 1.55 + 0.011 \cdot \sigma_c^{1.07} \cdot sin^2 (\alpha + i) \right] \sigma_c^{-0.14} \cdot (0.85 + 0.45 \tan \theta)$$  \hspace{1cm} (6)

where, $T_0$ is maximum shear resistance, $P_t$ is maximum tensile load of the bolt, $\theta$ is angle of friction, and $\alpha$ is bolt inclination.

Pellet and Egger (1996) developed an analytical model to investigate the shear strength of an untensioned fully grouted rock bolts. The bolt's behaviour in interaction between axial and shear forces and its large plastic displacements were measured. The behaviour of rock bolt in elastic and plastic zones are as shown in Figure 6. The Tresca criterion was used to model the bolt in failure under the combination of shear and axial loads. The single shear test was the methodology for this study, and the shear force acting at the shear plane can be determined by:
\[ Q_{of} = \frac{nD_b^2}{b} \sigma_{ec} \sqrt{1 - 16 \left( \frac{N_{of}}{nD_b^2 \sigma_{ec}} \right)^2} \]  

(7)

where, \( Q_{of} \) is shear force acting at shear plane at failure of the bolt, \( P_u \) is the bearing capacity of the grout or rock, \( D_b \) is diameter of the bolt, \( \sigma_{el} \) is the yield stress of the bolt, \( N_{oe} \) is axial force acting at shear plane at the yield stress of the bolt, \( \sigma_{ec} \) is failure stress of the bolt and \( N_{of} \) is axial force acting at shear plane at failure of the bolt.

Aziz et al., (2015) conducted a series of double shear tests on seven different types of cable bolt subjected to different amount of pre-tension load to determine the shear strength of pre-tensioned fully grouted cable bolts (Figure 7). The total shear strength was measured as the combination of shear strength of cable bolts and the friction between concrete blocks.

Aziz et al., (2015) also proposed a mathematical model to determine the shear strength of cable bolts using the combination of Mohr Coulomb criterion and Fourier series scheme. This model is capable of determining the peak shear strength of the cable bolt (Aziz et al., 2015). The proposed model is:

\[ \tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos \left( \frac{2n\pi}{T} \right) \cos \left[ \frac{-4a_2 + \sqrt{16a_2^2 - 48a_1a_3 + 144a_3^2}}{24a_3} \right] \right) \tan(\phi) + c \]  

(8)

where, \( \tau \) is the shear stress, \( S \) is the shear load, \( C \) is cohesion, \( a_n \) is Fourier Coefficient, \( n \) is the number of Fourier Coefficient, which is considered between 0 and 3, \( u \) is the shear displacement and \( T \) is the shearing length.
The test apparatus, sample preparation and experimental plan use by Aziz et al., (2015) were as follows:

Three concrete blocks with cross sectional area of 300 x 300 mm$^2$ made the double shear assembly. The length of the corner concrete blocks was 300 mm while it was 450 mm for the middle one. After curing, the concrete blocks had a compressive strength of 40 MPa and they were kept wet for 28 days for curing purposes. The concrete blocks were positioned in the steel moulds. During pre-tensioning and shearing test the axial loads were applied on the cable bolt. Therefore, two 60 t load cells were incorporated in order to monitor the axial loads. Two sets of barrels and wedge were installed to retain the cable bolt in tension. Depending on the test requirements, chemical resin or cementitious grouts were injected to the holes on top of the concrete blocks. Seven days after pre-tensioning the cable bolt and the grout injection, double shear tests were conducted with the 500 t machine at the rock mechanics laboratory of University of Wollongong. The rate of shear displacement was set by the digital controller as 1 mm/min. The shear load was applied to the middle concrete block to moving in a vertical direction by using a hydraulic jack located on top of the instrument. The data taker recorded the amount of shear and normal loads and shear displacement.

More than 10 double shear tests were conducted but in this paper the result of two tests will be analysed and compared with the proposed models. Two tests were conducted on the plain superstrand and spiral superstrand cable bolts with pre-tension loads of 25 t. Table 1 represents the test plans.

<table>
<thead>
<tr>
<th>Product name</th>
<th>Cable Ø (mm)</th>
<th>Wire geometry</th>
<th>Cable cross-section</th>
<th>Cable geometry</th>
<th>Drill bit (mm)</th>
<th>Bonding agent</th>
<th>Pre-tension load (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spiral superstrand</td>
<td>21.8</td>
<td>Spiral</td>
<td>19 wire, PC strand</td>
<td>Non-birdcaged</td>
<td>28</td>
<td>Oil based resin</td>
<td>25</td>
</tr>
<tr>
<td>Plain superstrand</td>
<td>21.8</td>
<td>Plain</td>
<td>19 wire, PC strand</td>
<td>Non-birdcaged</td>
<td>28</td>
<td>Oil based resin</td>
<td>25</td>
</tr>
</tbody>
</table>

As Figure 8 shows the peak shear load for the plain and spiral superstrand cable bolts were 1258 kN and 1115 kN, respectively. The reason is that the cross sectional area of the spiral superstrand cable is reduced compared to the plain superstrand cable. It shows that the cable behaviour starts with the elastic stage up to the yield point. Then, the strain softening stage starts up to the peak shear load and after this stage, the cable wires start to break. The drop in the shear strength depends on the size of cable wires.

**ANALYSIS**

This part consists of the comparison between the test data and model results. This result helps to identify the model which is in more agreement with experimental results in reality. The experimental results of shear test of two types of cable bolts, plain wire superstrand and spiral wire superstrand, will be compared with proposed analytical models. The properties of these two types of cable bolts subjected to shearing are summarised in Table 2. The $\sigma_c$ of concrete blocks were 40 MPa and $\sigma_y$ (yield stress) of bolts was 1677.3 MPa.

Tables 3 and 4 show the shear load measured by different models and the variation between their prediction results and the test result. The results demonstrate that the model developed by Aziz et al., (2015) has the least amount of variation with test results. Therefore, it is concluded that this model is
significantly suitable to calculate the peak shear load generated on pre-tensioned fully grouted cable bolts. Additionally, the model by Hass (1976), Bjurstrom (1974) and Pellet and Egger (1990) calculates shear load with less than 20% variation while for Aziz et al., (2015), it is far less.

Figure 8: Test results of plain and spiral superstrand cable bolts

Table 2: Properties of plain wire Superstrand and spiral wire Superstrand cable bolts

<table>
<thead>
<tr>
<th>Performance Data</th>
<th>Unit</th>
<th>Plain wire Superstrand</th>
<th>Spiral wire Superstrand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Load</td>
<td>kN</td>
<td>525</td>
<td>525</td>
</tr>
<tr>
<td>UTS</td>
<td>kN</td>
<td>590</td>
<td>573</td>
</tr>
<tr>
<td>Strand diameter</td>
<td>mm</td>
<td>21.8</td>
<td>21.8</td>
</tr>
<tr>
<td>C.S.A.</td>
<td>mm²</td>
<td>313</td>
<td>277</td>
</tr>
</tbody>
</table>

Table 3 Measured shear load for different models for plain wire Superstrand cable bolts

<table>
<thead>
<tr>
<th>Model</th>
<th>Shear load (kN)</th>
<th>Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dulacska (1972)</td>
<td>54.16</td>
<td>91.38</td>
</tr>
<tr>
<td>Bjurstrom (1974)</td>
<td>520</td>
<td>17.19</td>
</tr>
<tr>
<td>Haas (1976)</td>
<td>682.65</td>
<td>8.6</td>
</tr>
<tr>
<td>Spang and Egger (1990)</td>
<td>553.87</td>
<td>13.38</td>
</tr>
<tr>
<td>Pellet and Egger (1996)</td>
<td>227.5</td>
<td>63.8</td>
</tr>
<tr>
<td>Aziz et al., (2015)</td>
<td>630.07</td>
<td>3.3</td>
</tr>
<tr>
<td>Test result (Plain superstrand cable bolt)</td>
<td>628</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 4 Measured shear load for different models for spiral wire Superstrand cable bolts

<table>
<thead>
<tr>
<th>Model</th>
<th>Shear load (kN)</th>
<th>Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dulacska (1972)</td>
<td>54.16</td>
<td>90.29</td>
</tr>
<tr>
<td>Bjurstrom (1974)</td>
<td>490.17</td>
<td>12.08</td>
</tr>
<tr>
<td>Haas (1976)</td>
<td>653.57</td>
<td>17.23</td>
</tr>
<tr>
<td>Spang and Egger (1990)</td>
<td>738.8</td>
<td>32.52</td>
</tr>
<tr>
<td>Pellet and Egger (1996)</td>
<td>125.71</td>
<td>77.45</td>
</tr>
<tr>
<td>Test result (Spiral superstrand cable bolt)</td>
<td>557.5</td>
<td>-</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The use of cable bolts as a secondary support system in the mining industry is on an increasing trend. Therefore, it is essential to calculate the performance of cable bolts subjected to shear load. This has been a topic of interest for many researchers and different mathematical models have been proposed to determine the shear strength of cable bolts. The comparison between a number of these models demonstrated that the model proposed by Aziz et al., (2015) is in agreement with experimental result for plain and spiral superstrand cable bolts with 25 t pre-tension loads with less than 4% variation. Thus, it is rational to use this model to investigate the shear strength of a pre-tensioned fully grouted cable bolt. Moreover, Bjurstrom (1974) and Haas (1976) also proposed models that are useful but provide results which are less accurate compared to Aziz et al., (2015).

REFERENCES

Fuller, P G and O’Grady, P, 1993. Flexibolt flexible roof bolts: a new concept for strata control, In the 12th Conference on Ground Control in Mining, Morgantown, WV, USA, pp. 24-34.
A FOLLOW UP TO STUDY THE BEHAVIOUR OF CABLE BOLTS IN SHEAR: EXPERIMENTAL STUDY AND MATHEMATICAL MODELLING

Naj Aziz, Ali Mirza, Jan Nemcik, Haleh Rasekh and Xuwei Li

ABSTRACT: A mathematical model which is able to determine the pure shear strength of cable bolt was introduced and calibrated for various conditions. The proposed equation is developed based on the Fourier series concept and a linear relationship between shear and normal forces generated during cable bolt shearing. The conventional double shearing instrument was up to date to evaluate the pure shear strength of cable bolt by removing the contribution of concrete block frictional force. The experimental data obtained from the revised double shearing instrument was in good agreement with the prediction of the proposed mode.

INTRODUCTION

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

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

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

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

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

Craig and Aziz (2010) studied shear behavior of cable bolts using a large scale double shear instrument as shown in Figure 4. It consisted of three concrete blocks with outer two cube blocks of 300 x300 and 300 mm sides and the middle block of 300 x 300 x 450 mm sides. The concrete blocks were cast in the steel frame of the double shear apparatus, and before the concrete could be cast, a 20 mm diameter conduit pipe was placed through the pre-cut holes in the centre of the wooden ends and galvanised steel separators of the mould. The cable bolt was eventually encapsulated in the concrete blocks using an appropriate grout.
An experimental study on the shear performance of plain and spiral cable bolts was carried out by Aziz et al., (2014) using double shear apparatus. Other studies on shear behavior of cable bolts incorporating the same experimental instrument, were reported by Aziz et al., (2015a), Rasekh et al., (2015) and Li et al., (2015a and b).

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

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

**MATHEMATICAL MODELING**

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

\[ S - N \tan(\varphi) - c = 0 \]  

(1)

where, S is the shear load, N is the normal load, \( \varphi \) is the friction angle and c is the cohesion.

The Fourier series concept is applied to replicate the variation of the normal load against shear displacement. Fourier series is a mathematical technique incorporated to solve a large variety of engineering problems mainly adopting the principle of superposition:

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

(2a)

\[ a_n = \frac{2}{T} \int_0^T \sigma_n \cos\left(\frac{2n \pi u}{T}\right) du \]  

(2b)
\[ b_n = \frac{2}{T} \int_0^T \sigma_n \cos \left( \frac{2n \pi u}{T} \right) du \]  

(2c)

where, \( a_n \) and \( b_n \) are Fourier coefficients, \( n \) is the number of Fourier coefficient, \( u \) is the shear displacement and \( T \) is the shearing length.

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

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

(3)

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

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

(4)

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

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

(5)

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

\[
S_p = \left( \frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos \left( \frac{2n \pi T}{2\pi} \times \cos^{-1} \left[ \frac{-4a_2 + \sqrt{16a_2^2 - 48a_1a_3 + 144a_3^2}}{24a_3} \right] \right) \right) \tan(\phi) + c
\]

(6)

The model coefficients including Fourier coefficients \( (a_n) \), cohesion \( (C) \) and angle of friction \( (\phi) \) are determined according to the measured data for various test conditions such as the cable type and pre-tension. Generally, the values of Fourier coefficients showed a decreasing trend with increasing the number of Fourier coefficients.

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

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

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

(7)
where, $\phi_b$ is the concrete surface basic friction angle determined by tilt testing.

Deducting equation 7 from equation 6, the pure shear strength of cable bolt ($S^b_p$) is obtained as:

$$
S^b_p = \left(\frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2n\pi}{T}T \cos^{-1}\left[\frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_3^2}}{24a_3}\right]ight)\right)[\tan(\phi) - \tan(\phi_b)] + c
$$

(8)

Concrete surface basic friction angle

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

Figure 5: test results of the concrete blocks sliding test

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

$$
S^b_p = \left(\frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2n\pi}{T}T \cos^{-1}\left[\frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_3^2}}{24a_3}\right]ight)\right)[\tan(\phi) - \tan(26.94°)] + c
$$

(9)
Model calibration

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

MODEL VERIFICATION

To verify the proposed equation for the pure shear strength of cable bolts, two new double shear tests were undertaken. In one test the concrete surfaces were maintained in contact with each other and in the other without, that is no frictional resistance. In both tests various parameters, such as pretension load, grout type and concrete strength were kept constant. To achieve cable bolt shearing without contact between concrete blocks, the double shear apparatus was modified by installing two lateral braces on each side of the assembly as shown in Figure 6a to impede subjecting normal load on concrete blocks during shearing. To further assure no friction between concrete blocks, a pair of Teflon sheets with negligible fiction coefficient was introduced between concrete joints as illustrated in Figure 6b. Table 2 compares the values of the pure shear strength of cable bolts obtained from the proposed equation and experiments. It is clear that the experimental test result is reasonably close with the mathematical equation modelling.

Figure 6: Up to date double shear instrument (a) the whole assembly inside compression machine (b) Teflon sheet layers between concrete blocks
**Table 1a: list of tested cables and the test environment Aziz et al., (2015b)**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product name</th>
<th>Cable ø (mm)</th>
<th>Wire geometry</th>
<th>Cable cross-section</th>
<th>Cable geometry</th>
<th>Drill bit (mm)</th>
<th>Bonding agent</th>
<th>Pre-tension load (kN)/Peak axial load</th>
<th>Peak shear load (kN)</th>
<th>[½ double shear]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Superstrand</td>
<td>21.8</td>
<td>Spiral</td>
<td>19 wire, PC strand</td>
<td>Non-birdcaged</td>
<td>28</td>
<td>Oil based resin</td>
<td>250</td>
<td>558</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Superstrand</td>
<td>21.8</td>
<td>Plain</td>
<td>19 wire, PC strand</td>
<td>Non-birdcaged</td>
<td>28</td>
<td>Oil based resin</td>
<td>250</td>
<td>628</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>TG</td>
<td>28</td>
<td>Spiral</td>
<td>9 wires, hollow centre</td>
<td>Non-birdcaged</td>
<td>42</td>
<td>TD80 Grout</td>
<td>250</td>
<td>604</td>
<td></td>
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<tr>
<td>4</td>
<td>SUMO</td>
<td>28</td>
<td>Spiral</td>
<td>9 wires, hollow centre</td>
<td>35mm birdcage</td>
<td>42</td>
<td>TD80 Grout</td>
<td>250</td>
<td>414</td>
<td></td>
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<tr>
<td>5</td>
<td>SUMO</td>
<td>28</td>
<td>Spiral</td>
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<td>35mm birdcage</td>
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<td>TD80 Grout</td>
<td>100</td>
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<td>6</td>
<td>Plain SUMO</td>
<td>28</td>
<td>Plain</td>
<td>9 wires, hollow centre</td>
<td>35mm birdcage</td>
<td>42</td>
<td>TD80 Grout</td>
<td>250</td>
<td>711</td>
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<td>7</td>
<td>Plain SUMO</td>
<td>28</td>
<td>Plain</td>
<td>9 wires, hollow centre</td>
<td>35mm birdcage</td>
<td>42</td>
<td>TD80 Grout</td>
<td>100</td>
<td>659</td>
<td></td>
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<tr>
<td>8</td>
<td>Gardford twin-strand</td>
<td>15.2</td>
<td>Plain</td>
<td>2 x 7 wire, PC strand</td>
<td>25mm Bulbs</td>
<td>55</td>
<td>BU100 Grout</td>
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<td>501</td>
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</table>

**Table 1b: Model coefficients for different types**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>(a_0)</th>
<th>(a_1)</th>
<th>(a_2)</th>
<th>(a_3)</th>
<th>(\phi)</th>
<th>(c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>624.59</td>
<td>-53.97</td>
<td>-28.72</td>
<td>25.73</td>
<td>52.13</td>
<td>8.82</td>
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<td>-157.50</td>
<td>42.83</td>
<td>-4.21</td>
<td>47.61</td>
<td>137.89</td>
</tr>
</tbody>
</table>

**Table 2a: Comparison between the proposed model for the pure shear strength of cable blots and experimental data**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product type</th>
<th>Bonding agent</th>
<th>Pre-tension load (kN)</th>
<th>Peak shear load per face (kN)</th>
<th>Friction between surfaces</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Plain</td>
<td>Strata binder HS</td>
<td>5</td>
<td>645.64</td>
<td>with</td>
</tr>
<tr>
<td>10</td>
<td>Plain</td>
<td>Strata binder HS</td>
<td>5</td>
<td>442.16</td>
<td>without</td>
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**Table 2b: Determination of shear load by the model**

<table>
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<tr>
<th>Test</th>
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<th>(a_1)</th>
<th>(a_2)</th>
<th>(a_3)</th>
<th>Model normal load (kN)</th>
<th>Tan (\phi)</th>
<th>Tan 26.94°</th>
<th>(c) (kN)</th>
<th>Measured peak shear load per face (kN)</th>
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<tbody>
<tr>
<td>Plain superstrand</td>
<td>324.77</td>
<td>182.37</td>
<td>18.65</td>
<td>3.04</td>
<td>366.316</td>
<td>1.47</td>
<td>0.508</td>
<td>88.61</td>
<td>441.006</td>
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<tr>
<td>with 5 kN pre-tension load</td>
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</table>
CONCLUSIONS

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

REFERENCES


LOAD TRANSFER CHARACTERISTICS OF PLAIN AND SPIRAL CABLE BOLTS TESTED IN NEW NON ROTATING PULL TESTING APPARATUS

Naj Aziz, Ali Mirza, Jan Nemcik, Xuwei Li, Haleh Rasekh and Gaofeng Wang

ABSTRACT: The load transfer mechanisms of cable bolts differ from normal rebar bolts. Cable bolts used in mines are basically steel strands with different constructions depending on the number of wires or elements and the way that these elements are laid. Tendon bolts (rebar and cable) are normally evaluated for strength and load transfer properties. The strength of tendon can be carried out by tensile failure tests, while the load transfer strength is evaluated by pull and shear strength tests. Short Encapsulation Pull Testing (SEPT) is used to study of the load transfer capacities of tendons, and can be undertaken both in the laboratory and in situ. A new apparatus known as Minova Axially Split Embedment Apparatus (MASEA) was used to study load-displacement characteristics of smooth versus spiral profile cable bolts. Minova Stratabinder grout was used for encapsulating 400 mm long 19 wire 22 mm diameter superstrand cable in the embedment units. The anchorage of the cable on two sides of the embedment apparatus were intentionally installed at different lengths, to allow the cable to be pulled out from one side of the anchorage. The spiral wire strand cable bolts achieved higher peak pull-out load at minimum displacement in comparison with smooth surface wire strand. The peak pull out force increased with the age of encapsulation grout. The MASEA was easier to assemble and test at a short period of time, thus allowing quick and repeated tests to be undertaken.

INTRODUCTION

For several decades now, cable bolting systems have been used for ground reinforcement and stabilisation in mines. Initially cable bolts were used for surface stabilisation structures such as dams and slopes, prior to their adoption in mines (Gillott and Mieville 1964). The use of cable bolts in underground mines initially began in metal mines and later on in coal mines. There are currently more that dozen types of cable bolts, classified into five main categories used in Australian mines. These are; (a) smooth or plain surface cable bolts; b) bulbed; c) nut caged; d) spiral and indented cable bolts; e) a mix of plain and spiral cable bolts. With the exception of Garford twin cables, most cable bolts used in Australian coal mines are made of seven, nine and 19 wire constructions. The 19 wire strand is of Warrington Seal Construction. The seven wire cables have six outer wires wrapped around the central core wire, which is known as the centre or king wire. However, the 19 wire cable has two layers of wires consisting of nine 5 mm diameter outer wires and; nine 3 mm inner layer wires, all wrapped or laid around the 7 mm inner or king wire. Recently a new nine wire cable was introduced to the Australian mines, which consisted of alternate smooth and spiral wires.

For a cable bolt support system to be effective, the loads have to be successfully transferred from the rock to the cable through the grouting materials. The increase in axial forces within the cable occurs during the movement of the rock mass at shear planes and bed separations. Thus the anchorage applied in the borehole, can be enhanced by various strand’s outer wires surface roughness such as indentations and spiral profiles as well as the presence of birdcages or bulbs.

During installation of cable bolts chemical resin grouts, cementitious grouts or a combination of both are used. The main method of assessing the long tendons performance is by evaluating both tensile and shear performance.

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The early interest in assessing the performance of cable bolts dates back to the work of Fuller and Cox (1975). Since then there has been a growing number of testing techniques and procedures reported in various publications because of the increased variety of cable bolt design and size. The current research on cable bolt assessment used as secondary support system.

The earliest method of determining the load transfer capacity of cable bolts was by encapsulating one end of the cable in a steel tube, while the other free end was to be used to pull the cable out using a tensile testing machine. This system was later extended to Double Embedment Pull Test (DEPT). The pull testing with double embedment installation was mostly used for tensile failure test rather than load transfer studies, as reported by Clifford et al., 2001. This methodology of testing was subsequently adopted in the British Standard BS 7861- Part 2 (1997) and later in amended edition of BS 7861- part 2 (2007). This suggested the double embedment method for cable pull testing to failure, whereby a suitable length of the cable was installed in embedment tubes with an internal diameter of 35 mm and outside diameter of 63.5 mm. The internal surface of each tube section was machined to a 2 mm pitch, 1 mm deep thread to prevent failure at the grout tube boundary. Two tubes, each 450 mm in length were used to install the end sections of cable bolt in each tube, which were butted together. The DEPT has been used both for the evaluation of the cable ultimate strength as well as for load transfer capacity studies.

Thomas (2012) reported on the load transfer of post-groutable cable bolts and described the fundamental aspects of cable bolt load transfer and testing procedures. He critically reviewed various methods of cable bolt pull out tests and undertook a series of pull tests on 14 cables types using a modified version of the Laboratory Short Encapsulation Pull Test (LSEPT) apparatus initially developed by Clifford et al., (2001). Thomas (2012) reported on variations in load displacement profiles between plain and profiles surfaces of the different cable bolts.

Thomas (2012) described the fundamental aspects of cable bolt load transfer, and testing procedures, focusing on the latest innovation of the testing systems applied and on their significance. Citing the study undertaken by Clifford et al., 2002, which allowed an amount of assessment of the grout to rock interface and hole rifling, that better simulated the underground environment, however Thomas (2012), questioned the use of high 10 MPa confining pressure of the biaxial force applied on the rock anchors, as being inconsistent with the underground ground pressure environment. Subsequently Thomas (2012) modified the Clifford developed system by replacing the biaxial pressure cell with a thick walled steel cylinder and the whole assembly was locked up together with an anti-rotation device to prevent the cable from unwinding out of the core when the pull load was applied as shown in Figure 1. Other points for noting were:

- The diameter of the sandstone medium was 142 mm and the UCS values ranged between 19 to 25 MPa, and
- A barrel and wedge was embedded in the cementious or resin grout inside the concrete column inside the steel tube.

Hagan et al, (2014), in the ACARP project C2010, reported a chronological review of different pull testing techniques of since the mid-1980's and extended the non-rotating cable concept developed by Thomas (2012) during pull testing to include:

1. Testing of the cables in concrete cylinders.
2. Applying confining pressure by enclosing the concrete cylinders in two section steel cylinders.

Studies were subsequently undertaken to gauge the sensitivity of several parameters; the strength of the concreted that is used for testing; the diameter of the borehole size and thickness of grout encapsulation in relation to the concrete strength. Further studies carried out include the following:
• The development of an axial loading test procedure for cable bolts used in Australian underground mine,
• Development of a new laboratory-scale test facility for pull testing of various cables,
• Optimisation of the concrete cylinder size that lead to the optimisations of pull testing of various cable bolts and

![Figure 1](image1.png)

**Figure 1:** A modified Laboratory short encapsulation pull test (Thomas 2012)

Further amendments of single embedment length pull-out tests include:

- Cable testing in unconfined as well as confined condition,
- Confined concrete sample diameter increased from previously used 142 mm to 300 mm, the latter being the most suitable size,
- The concrete sample enclosed in a steel cylinder (axially split) and assembled by bolting together two half cylinder making it easier to de assemble.

Figure 2 shows the UNSW assembled cable pull testing facility (Hagan et al., 2014) and Chen et al., 2014).

![Figure 2](image2.png)

**Figure 2:** UNSW LSEPT pull testing apparatus (Chen et al., 2014)
From various tests procedures reviewed it is obvious that in each test described, a significant amount of wastage occurs in terms of material used, and cost, because of:

- The need for steel tubes with regard to testing using single or double embedment tubes. These tubes are only used once, thus multiple tests require more tubes.
- The need for availability of rock samples for cable bolt installations, requiring sample preparation for cable installation, and
- In the case of the latest short encapsulation testing as developed by Hagan, requiring 300 mm diameter concrete test samples and consumable anchor tubes.

In this paper a new instrumentation is described that eliminates the need for rock or concrete samples and other consumables. The system will permit repeated test to be undertaken economically and at a much faster rate, and the new system can be further modified to allow various diameter cables to be tested, and the system is particularly suited for comparatives cable bolt design tests.

AXIALLY SPLIT DOUBLE EMBEDMENT PULL TESTING

Design

Figure 3 shows a detailed drawing of the axially split SEPT apparatus. Developed by Minova Australia, the apparatus has two embedment sections, with each section consisting of two half blocks of steel with semicircular holes carved out in the middle. Two sections are butted together and bolted tight using eight Allen socket head bolts, 50mm long and 8 mm in diameter. Thus, the central hole becomes 30 mm diameter hole 250 mm long. The internal surface of the central hole has grooves 3mm deep and spaced 10 mm apart as shown in the detailed design in Figure 3. The objective is to allow effective anchorage of the resin/grout to the outer hole wall. A rectangular 10 mm thick steel sleeve, inserted on the assembled embedment apparatus ensured non-rotation of the anchored cable during pull out testing. A 100 mm long window on one side of the sleeve was cut for viewing of the pulled out cable as shown in Figure 4. The cable anchorage is achieved using chemical resin or cementitious grout and re-use of the capsule is possible after each completed test. The removal of grout post-test and cleaning of the steel capsule for re-use was found to be easier with grouts in comparison to chemical resin.

![Figure 3: Detail drawing of the Minova axially split SEPT assembly](image-url)
Pull testing

The objective of the study was to compare the pulling force between plain and spirally profiled cables. In this study 22 mm diameter 19 wire super-strand cable bolt sections were tested. Both plain and spiral cables were used and each tested cable piece consisted of 400 mm long sections with free ends welded to ensure the wiring assembly remains intact during pulling. Each cable was anchored in the steel sleeves at different lengths. The aim was to let the cable to be pulled-out from the shorter side sleeve leaving the other side to act as intact anchorage. Accordingly, one side of the cable was encapsulated to a depth of 230 mm and the other at 170 mm. This arrangement was necessary to let cable to be pulled out from one side only to gain better understanding of pull out behaviour between plain and spirally profiled cables. Figure 6 shows a post-test view of the cable in opened apparatus.

Results and analysis:

Figure 5 shows the load-displacement graphs of six tests. Tested cables were encapsulated in the holders using the same resin of various ages. Ages of encapsulated grout were; four days, one week, and one month. Figure 6 shows the view of the split assembly after pull testing. Table 1 shows the initial peak load of various tested cables and optimum displacement.

![Figure 4: Axially split double embedment pull testing apparatus](image)

![Figure 5: Load – displacement graphs of pull testing of cables at different encapsulating grout ages](image)
Figure 6: Post-test two halves of the pull out apparatus with encapsulated cable bolt

Table 1: Pulled out peak load values of tested cables with grout ages of four days, one week and one month of encapsulations, for both smooth and spiral wired cable strands

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Four days encap.</th>
<th>One week old encap</th>
<th>One month old grout</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain</td>
<td>Spiral</td>
<td>Plain</td>
</tr>
<tr>
<td>Peak Load (kN)</td>
<td>115.0</td>
<td>148.2</td>
<td>150.8</td>
</tr>
<tr>
<td>Displacement (mm)</td>
<td>50</td>
<td>2.5</td>
<td>46.0</td>
</tr>
<tr>
<td>Bond strength (kN/mm)</td>
<td>0.676</td>
<td>0.87</td>
<td>0.887</td>
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</table>

The load - displacement graphs of six tests shown in Figure 5 indicate the following:

1) When pulling the spiral cables, all peak loads occurred at displacements of less than 5 mm however, the respective peak loads for plain cables were significantly greater.
2) The displacements at the peak loads were generally higher for spiral wire cables,
3) The profiles of spiral and plain cable pull loads as well as their respective displacement were in agreement with the load-displacement profiles reported by Thomas (2012) for tests made in sandstone blocks. The incorporation of the steel sleeve on the pulling apparatus shown in Figure 5 clearly demonstrates its effectiveness in elimination of cable rotation during pull testing.
4) As expected, the bond strength of the tested samples was noted to increase with encapsulation grout curing age. The peak load per millimeter of the encapsulated plain cable length ranged between 0.676 kN/mm for four day grout cure to 1.029 kN/mm for one month old cure. Similarly for the spiral profile these values ranged between 0.87 kN/mm and 1.104 kN/mm respectively. These values were not much different from the test results of Thomas carried out on 19 mm diameter Hilti cables of 1.10 kN /mm for spiral cable and 0.672 kN/mm for the plain cable respectively, bearing in mind that; a) the embedded cable length conducted by Thomas (2012) in sandstone block was 320 mm; and b) the resin used was different from the Minova Mix and Pour resin used in this Peak load achieved with plain cable bolts occurred at greater displacement, irrespective of the grout or resin installation age. The profiles of load displacement are in agreement with the results of Thomas (2012).
5) The use of the steel encapsulation frame allowed repeatable, faster and economical tests.
6) The MASEA test apparatus was designed for pull testing of limited diameter tendons. Figure 7 shows an alternative apparatus for pull testing of different diameter tendons. This new apparatus is named as Multi Diameter Laboratory Short Encapsulation test (MDLSET) apparatus. This instrument will permit pull out tests of cables of different diameters.
7) While the use of steel frame is no substitute to testing of the cables in rocks and in composite materials such as concrete, nevertheless, test results are consistent and with similar results reported by Thomas in sandstone and steel frame.
A new Minova pull out instrument has been developed for testing cable bolts. It is simple in design and construction. Its main benefit is that it is a fast method which requires no additional testing material other than the resin or grout for cable encapsulation. The current instrument is designed to suit rock bolts and cable bolts of 22-24 mm diameter. However the system can be extended to permit testing of cables of any diameter and bulbs. This has been achieved by enlarging the system and incorporating separate sleeves of the internal grooves fitted in to outer shelves of the instrument and is known as MDLSET.

Using the MLSEPT instrument it was found that spiral cables pull loads were higher than and occurred at shorter displacement in comparison with smooth cables. Also the peak loads were found to increase with the curing time age of encapsulation irrespective of the cable type.

ACKNOWLEDGEMENT

The pull testing instrument described was designed and constructed by Minova Australia and has been used to carry out various tests to prove the viability of the equipment for fast and economical in application. The authors are grateful to Minova for permission to use this instrument for various testing. The new MDLSET type design system as reported in this paper has been developed and proved successful.

REFERENCES


SAMPLE SIZE AND SAMPLE STRENGTH EFFECTS ON TESTING THE PERFORMANCE OF CABLE BOLTS

Hao Zhai, Paul Hagan and Danqi Li

ABSTRACT: This paper presents the results of a study on the effect of test sample diameter on the peak load carrying capacity of cable bolts in varying conditions. It was previously found that peak load varies with the diameter of test sample up to 300 mm and further that confinement has a significant impact on load. In this study test samples were varied over a larger range of diameters up to 500 mm in test samples having strengths of 32 MPa and 66 MPa using a plain strand Superstrand cable bolt and a nutcage high capacity MW9 cable bolt. The test work confirmed that there are differences in anchorage performance in material of different strength between the two cable bolts and importantly that a 300 mm diameter sample size is required when comparing different types of cable bolts design.

INTRODUCTION

Cable bolts are widely used for ground support in both the civil construction and mining industries. They are used to prevent movements of joints by transferring loads from the failing side of the discontinuity plane to the intact side of the discontinuity plane (Hutchinson and Diederichs 1996).

Considering the importance of cable bolts in this role, a systematic method of determining and comparing the performance of different types of cable bolts is important in optimising the selection in differing ground conditions. Over the years, a variety of testing methods such as “split-pull/push” test, single embedment test, double embedment test and Laboratory Short Encapsulation Pull Test (LSEPT) have been developed to better understand the load transfer mechanism of cable bolts (Hagan et al., 2014). The study by Thomas (2012) revealed several deficiencies in the LSEPT developed in the UK that had been adopted as the industry standard test method. As a result, an Australian Coal Association Research Program (ACARP) sponsored project was undertaken at UNSW to develop a new cable bolt testing facility.

In testing cable bolts, the primary variables include the design and form of cable bolt and properties and dimensions of the test sample in which the cable bolt is anchored. Previous research reported by Rajaie (1990) using a plain strand cable bolt found the peak load increased with the diameter of the test sample up to around 200 mm and thereafter remained largely unchanged. With the development of high capacity, modified cable bolts since that time, it was not known whether this diameter was still applicable or whether sample strength had any effect on anchorage performance. Recent studies have been undertaken on the basis that test results might be compromised if load capacity of a cable bolt was a function of test sample size used in anchoring the cable. This paper examines the influence of changes in the diameter of the test sample on the peak pull-out load in two strengths of test samples and two types of cable bolts. A series of pull tests was undertaken with samples varying over a range of diameters up to 500 mm in high strength and low strength materials when using a plain strand Superstrand cable bolt and a nutcage high capacity MW9 cable bolt.

BACKGROUND AND OBJECTIVES

An earlier study by Rajaie (1990) involved 295 tests with test samples in the unconfined state, that is test sample cylinders without any lateral constraint with diameters ranging between 100 mm and 300 mm. A 15 mm diameter plain strand cable bolt was used in the tests with grout used as the binding material for cable bolt installation. As is shown in Figure 1, the peak load carrying capacity increased with sample diameter until it reached a plateau at around 200 mm. Beyond this diameter there was little further change in load. Based on this observation, Rajaie recommended that test sample diameter used for pull out tests should be at least 250 mm.
Subsequently, there have been many developments in cable bolt design leading to large diameter high capacity cable bolts and it is conjectured that the minimum diameter of test sample recommended by Rajaie may no longer be applicable due to the much higher stresses induced in the test sample (Hagan and Chen 2015). Holden and Hagan (2014) replicated Rajaie’s work using a high capacity bulbed cable bolt and confirmed that the peak load carrying capacity continued to increase with test sample diameters in excess of 200 mm.

Ur-Rahman et al., (2015) reported a subsequent study on the peak load carrying capacity and the sample diameter relationship using a bulbed Sumo cable bolt with a diameter of 28 mm. As shown in Figure 2, the threshold value of sample diameter was found to be much larger at 300 mm. It should be mentioned that tests at some diameters were not replicated to account for variability and hence there was some scatter in results. A major development since Rajaie’s study is the introduction of confinement to test samples. The LSEPT test method uses a biaxial cell to apply a constant stress condition during testing. More recently Hagan and Chen (2014) reported on the use of a steel barrel or cylinder to simulate the confinement of a rock mass that provides passive confinement in reaction to any stress induced in the test sample during or as a consequence of load being applied to the cable bolt. However, the stress state in the surrounding rock mass may not be consistent under loading by a cable bolt (Thomas 2012). To overcome this issue, a split steel cylinder was used where the bolts joining the two halves of the split cylinder were torqued to the same level to provide a consistent level of sample confinement.
The main objectives of this study were to:

- determine whether there were any differences in the peak load/sample diameter relationship for plain Superstrand and nutcaged MW9 cable bolts;
- compare the results for peak load in test samples of different strength; and
- recommend a minimum test sample diameter for pull-out tests applicable over the range of cable bolts currently in use.

**METHODOLOGY**

The testing methodology developed by Ur-Rahman et al., (2015) was used as the basis of this study. However, the embedment length, rifling intervals and the level of confinement were altered in line with the development of the UNSW pull test facility. Furthermore, polyester resin was replaced by high fluidity grout to ensure consistency in cable bolt installation.

**Sample preparation**

An MW9 nutcage cable bolt and a Superstrand plain cable bolt were selected to represent a high and low performance bolt respectively. Both types of cable bolt used in the tests were 1200 mm in length of which 270 mm was grouted in the test sample and 930 mm was left free for gripping as shown in Figure 3. For the MW9 cable bolt, the nutcage section spanned nearly 180 mm. The centre of the nutcage was positioned 140 mm from one end of the cable bolt leaving some 50 mm of unmodified cable bolt. The Superstrand cable bolt is 21.8 mm in diameter and has a solid king wire (the central wire) whereas the MW9 diameter is 32 mm in the plain section and 36 mm in the nutcaged section. The king wire in the MW9 is hollow to allow for grout injection in the field.

![Figure 3: Designs of Superstrand (upper) and MW9 (lower) cable bolts](image)

The test samples were prepared as a bulk-pour from an aggri-truck to ensure consistency in material properties. The high strength and low strength samples were cast in two batches. The cementitious material was poured directly into fibre glass and cardboard compound casting cylinders each having a height of 320 mm and inner diameters ranging from 200 mm to 500 mm. To create smaller 150 mm diameter test samples, casting moulds were made from PVC pipe. Pre-split lines were cut along the length of the PVC pipes to allow for easy demoulding sealed on the inside with duct tape to prevent leakage. The material was left to cure for a minimum of 28 days.

The casting moulds were glued onto 2 m by 2 m Medium Density Fibre Boards (MDF Board) using waterproof silicon glue as show in Figure 4. PVC pipes with 28 mm and 42 mm outer diameters were used as the rifled borehole moulds for Superstrand and MW9 cable bolt respectively. As shown in Figure 5a, a 5 mm diameter soft plastic tube was wound around the PVC pipe with a 20 mm pitch to promote the interlocking between grout and rock. The plastic rifling tubes were mounted on the PVC moulds by medium adhesive strength hot glues to ensure easy detachment in demoulding. The PVC
tubes were filled with foam on one side of the rifling tube to increase the contact surface area for glue application.

![Figure 4: Assembled casting moulds](image)

**Figure 4: Assembled casting moulds**

![Figure 5: a) Assembled PVC tube used as mould to create borehole rifling effect (left) and b) foam fill tube (right)](image)

**Figure 5: a) Assembled PVC tube used as mould to create borehole rifling effect (left) and b) foam fill tube (right)**

When the casting moulds were fully assembled with the rifling tubes, the cement grout was poured directly into the moulds as shown in Figure 6. A mechanical vibrator was used to remove any trapped air bubbles.

![Figure 6: Test samples after pouring](image)

**Figure 6: Test samples after pouring**

After 24 hours, both the outer cardboard mould and inner PVC pipe were removed and each test sample was then left to cure for 28 days fully submerged in tap water as shown in Figure 7. The top surface of each sample was covered by wet rugs which were moisturized on a daily basis.
As the borehole was cast to the full length of the test sample, the borehole was sufficiently backfilled with a cement mixture to leave a remaining 270 mm length for grouting of the cable bolt.

Both the Superstrand and MW9 cable bolts were installed into fully cured samples after at least 24 hours of backfilling. The same 60 MPa grout strength using a 0.42 water to cement (w:c) ratio was used as the binding material in both the strong and weak test samples. The cable bolt installation involved the following steps.

1. Insert the cable bolt into the fully cured sample;
2. Pour well mixed grout with 0.45 w:c ratio in the annulus between the cable bolt and the borehole wall;
3. Shake and rotate the cable bolt to remove trapped air inside the grout;
4. Support the cable bolt with alignment clamps and metal plates.

An example of cable bolt installation is shown in Figure 8.

Confinement of the test sample was by means of a split steel cylinder assembled using four bolts as shown in Figure 9. The cylinder was designed to allow for a 10 mm annulus between the test sample and the steel cylinder that was backfilled with general purpose cement with a w:c ratio of 0.45. A thin
layer of foam was inserted between faceplates to prevent mortar leakage. After 24 hours of curing, each of the four bolts were tightened with a micrometre torque wrench to 40 N\(\cdot\)m.

![Image of assembled split steel cylinder](image)

**Figure 9: Assembled split steel cylinder in place with sample ready for testing**

**Test procedures**

The testing system used for this test is presented in Figure 10 and 11. The test system comprises sections: the bolted section at the bottom and pull section at the top. During each test, a load was applied to the cable bolt by an hydraulic ram acting against a steel bearing plate on top of the test sample. The magnitude of applied load was measured using a load cell while displacement was measured using a Linear Variable Differential Transformer (LVDT) and a laser displacement sensor. During a test setup, the locations of the LVDT and laser displacement sensor had to be carefully adjusted because of the limited reading range.

![Diagram of testing system](image)

**Figure 10: Testing system design**
RESULTS AND ANALYSIS

Strength UCS tests

UCS strength tests were conducted on 50 mm cubic samples of the cable bolt grouting material and test samples one day before the pull tests began. Material strengths were derived from the average of five UCS test replications. Test results are presented in Table 1 with the mean values for strong and weak test samples as well as the grout being 66.2 MPa and 31.5 MPa respectively, representing a near 50% difference in material strength between the two samples. The strength of the material used to grout the cable bolt in the test sample was 53.7 MPa.

Nutcage MW9 cable bolt in strong test sample

Figure 12 shows the variation in measured peak load of the MW9 cable bolt with diameter of test sample using the strong cement material. In most cases, three test replications were undertaken at each level of test sample diameter. The data is a plot of the average of the three load measurements and indicates the range of standard deviation.
Table 1: Results of UCS strength tests

<table>
<thead>
<tr>
<th>Material</th>
<th>Test Number</th>
<th>UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strong test sample</td>
<td>1</td>
<td>65.95</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>66.52</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>66.17</td>
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Figure 12: Variation in peak load with sample diameter for MW9 in strong test samples

Over the range of diameters studied, there was a near 30% increase in load bearing capacity of the MW9 cable. The lowest load of 238 kN was attained with the 150 mm diameter sample, the smallest of test samples. Load increased with diameter until approximately 300 mm when it reached 305 kN at which point the load had effectively plateaued indicating sample size at his turning point of 300 mm no longer had any influence on the load bearing capacity of the cable bolt. In should be noted that only one sample was tested at the larger diameters because of issues in casting the test samples.
Nutcaged MW9 weak sample

![Graph showing variation in peak load with sample diameter for MW9 in weak test samples](image)

Figure 13: Variation in peak load with sample diameter for MW9 in weak test samples

Similar to the strong sample, there was a trend of increase in load with diameter in the weaker material up to approximately 300 mm as shown in Figure 13. The load at a diameter of 150 mm was 194 kN or nearly 80% of the load achieved in strong material. Again a turning point was reached at 300 mm with a load of 230 kN. The value of the peak load at this turning point in the lower strength test material was only 75% of that achieved in material nearly twice its strength.

Plain Superstrand strong samples

As shown in Figure 14, the range of sample diameter effecting peak load when using the Superstrand cable was much less extending to between 200 mm and 250 mm. The load at 150 mm was 145 kN or around 60% of the load achieved with the MW9 cable in the same strength sample. The maximum load that could be achieved was only 170 kN. This result of a much smaller diameter is consistent with the findings of Rajaie (1990).

![Graph showing variation in peak load with sample diameter for Superstrand in strong test samples](image)

Figure 14: Variation in peak load with sample diameter for Superstrand in strong test samples
Plain Superstrand weak samples

In tests with the Superstrand cable in weak material shown in Figure 15 a similar trend was evident with the turning point in diameter between 200 mm and 250 mm. The lower strength material did not appear to have any impact on the turning point diameter, only use of the lower capacity cable bolt. The value of the peak load was only 125 kN at and beyond the turning point. Interestingly, there seemed to be more variability in this combination of parameters than in any of the other tests with consistently more scatter in the mean values as well as larger standard deviations.

![Figure 15: Variation in peak load with sample diameter for Superstrand in weak test samples](image)

Analysis

The values for maximum load bearing capacity as well as the peak load for the Superstrand and MW9 cable bolts are compared in Figures 16 and 17 for strong and weak test samples respectively. It can be seen that different cable bolts have different sample diameters for turning points. For the Superstrand and MW9 cables, the turning points occurred at 200 mm and 300 mm respectively.

Rajaie (1990) stated in his work with plain strand cable bolts, the threshold value of sample diameter should be between 200 mm and 250 mm where the samples were tested in the unconfined condition. Rajaie’s finding is similar to the result for the Superstrand cables where the samples were tested in the confined condition.

![Figure 16: Comparison of cable bolt load bearing capacity in strong test samples](image)
The earlier work reported by Ur-Rahman et al., (2015) who used Sumo cable bolts, a similarly modified high capacity bolt, reported peak load continued to increase beyond a diameter of 250 mm and only plateaued at 300 mm as shown earlier in Figure 2. This suggests that when testing only plain strand cable bolts, a sample size of 250 mm is sufficient but when comparing performance with higher capacity bolts the minimum sample diameter needs to be at least 300 mm in order to eliminate any effect of sample size on measured performance.

Moreover, both the peak loads and turning point sample diameter of modified cable bolts are significantly greater than conventional plain cable bolts. Hence this emphasises the observation by Hagan and Chen (2015) that as cable bolts have evolved over recent decades, so to the parameters used in tests need to be reviewed because of the potential effects they can have on measured performance.

In comparing the performance of the two cable bolts shown in Figures 18 and 19, while strength of the test material had little perceptible effect on the turning point sample diameter, it had a marked effect on the peak load in each instance. Hence it is important when reporting test results to state the strength of the test sample used and as much as possible ensure consistent material properties when comparing the performance of different types of cable bolts.
CONCLUSIONS

It was found that within a certain range of test sample diameters used in performance testing of cable bolts, diameter has a marked effect on anchorage performance. In the case of the modified high capacity MW9 cable, a doubling in test sample diameter up to the turning point of 300 mm resulted in a nearly 30% increase in measured peak load. Beyond 300 mm there was no perceptible increase in load observed. With the plain strand Superstrand cable bolt, the range of diameters only extended up to the turning point of between 200 and 250 mm beyond which there was no further increase in measured load. It is likely that when load is applied to the cable bolt the stress induced within the test sample can result in premature failure of the confining material, hence limiting the load that can be applied to the cable bolt. As diameter of the test sample increases, so does the level of stress necessary to fail the test sample.

Strength of test sample had little or no effect on the turning point of diameter. With a doubling in material strength from 32 MPa to 66 MPa, the turning point diameter was unchanged for both types of cable bolt though it had a marked impact on the peak load that could be attained in each instance.

The results are consistent with the findings by Rajaie (1990) who used a plain strand cable bolt and the more recent work reported by Ur-Rahman et al., (2015) who used another type of modified bulbed cable bolt.

It is therefore recommended that the minimum diameter of test sample necessary when testing different types of cable bolts be at least 300 mm. Moreover it is important that as strength of the test sample has a marked impact on performance, controls are put in place to ensure consistent strength of the test samples. No one strength of test sample is recommended however, work by Hagan and Chen (2015) indicates failure mode varies with strength of test material and hence it plays an important role when understanding the differences in anchorage performance in different rock types such as coal and sandstone.

ACKNOWLEDGEMENTS

The authors would like to thank the support provided from the Australian Coal Association Research Program (ACARP), Megabolt Australia and Jennmar Australia without which this study would not have been possible. The authors also gratefully acknowledge the great assistances provided by Mr. Kanchana Gamage and Mr. Jianhang Chen.
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PRACTICAL INVESTIGATIONS INTO RESIN ANCHORED ROOF BOLTING PARAMETERS

Jacqui Purcell¹, Damon Vandermaat², Michael Callan³ and Peter Craig⁴

ABSTRACT: Resin bolt parameters, such as back pressure and gloving, and their effect on ground support system performance, remains one of the fundamental areas of rockbolt research. The majority of previous studies into resin bolting parameters have utilised various methodologies to investigate the effect of a singular parameter. Unfortunately, due to the variability in methodologies and the relatively narrow field of study of each research project, a holistic conclusion into the exact science behind various results is unable to be drawn. It is the focus of this research project to conduct a detailed and consistent testing program, which attempts to simulate real world conditions as closely as possible, in order to provide the industry with engineered roof bolting solutions to specific underground roof properties.

Recently published studies have implemented steel piping as a simulated borehole and have reported relatively high back-pressure measurements. As part of this project, testing conducted both underground and in a cement block, have recorded back-pressures in the range of 4-10 MPa, which are substantially lower than previous tests conducted in steel piping.

Rockbolts installed in an underground coal mine using a continuous miner have been over-cored, the core has been cut into 100 mm lengths and each sample has been push tested. After push testing, the samples from the top 300 mm of each bolt were inspected for gloving. Almost all of the recovered rockbolts experienced some degree of gloving within the top 300 mm of its length. The average severity of gloving within these specimens was found to be relatively minor. It was found that gloving can reduce load transfer by 4-6 kN per 10% of gloved surface area.

INTRODUCTION

Full or partial gloving of resin anchored rockbolts was first observed in the late 1980’s (Pettibone 1987), and has been an area of active research ever since. In a theoretical, perfect world, a correctly installed rockbolt will completely shred the film forming the two component resin capsule, facilitating correct mixing of the mastic and catalyst phases.

However, in reality, as the bolt is pushed up into the resin capsule, the rockbolt can act as a piston, the borehole in the rock as the cylinder and the resin as a pressurised fluid. The annulus between the rockbolt and borehole allows the pressure in the resin to be relieved as it flows down the side of the bolt. The pressure within the drill-hole is often referred to as ‘resin back-pressure’. The thrust applied to the bolt causes the resin capsule to swell, split and the film to be pressed against the wall of the drill-hole, resulting in gloving of the rockbolt (Campbell and Mould 2003).

Gloving is incomplete destruction (either partial or total) of the resin capsule film at the time of rockbolt installation (Pettibone 1987). This can result in a low friction plane of weakness along the resin/rock interface, which can potentially impact on the anchorage strength of the rockbolt.

It has been theorised that resin back-pressure could lead to hydraulic fracturing of rock within the top section of the borehole during bolt installation (Evans 2015 and Campbell and Mould 2003). It has also...
been previously theorised that pressures as low as 4 MPa are capable of causing hydraulic fracturing where the minor horizontal stress is in the same order of magnitude, forcing closed joints or partings to open (Campbell and Mould 2003). The opening of these fractures could provide a route for resin to escape, leading to resin loss and reduced encapsulation. This theorised phenomena is shown in Figure 1.

\[
\sigma_1 = 3\sigma_2 + S - P_i - P_o = 3\sigma_2 - P_r, \quad \text{where}
\]

- \(\sigma_1\) = major principal stress
- \(\sigma_2\) = intermediate principal stress
- \(S\) = hydraulic fracture (tensile) strength of the rock
- \(P_i\) = crack initiation pressure
- \(P_o\) = pore pressure
- \(P_r\) = crack re-opening pressure

The orientation of the minor horizontal stress governs the orientation along which a hydraulic fracture will propagate. The majority of coal mines in Australia have a vertical principal stress (\(\sigma_v\)) which is relatively accurately calculated via depth. Coal mines located at a depth of 200 – 400m below the surface would be expected to have 5-10 MPa horizontal stress. Stress redistribution around mined roadways will alter the principal stresses away from their virgin condition, causing notches and reliefs in various areas (relative to the direction of the virgin stress directions with respect to the roadway). For the sake of a first pass analysis of the required pressures for crack initiation however, virgin stress conditions will be implemented in the calculations. With \(\sigma_h\) typically being 1.2-2.5 times the vertical stress (Nemick et al., 2006) and \(\sigma_n\) typically being 0.6-2 times the vertical stress, and assuming tensile strengths of 1-10 MPa and pore pressure of 0 MPa (conservative), rearranging the equation above, examples of initiation pressure and reopening pressure have been calculated for real mine stress measurements, in Figure 2.
From this it can be seen that there is a vast range of pressures which can be expected to either initiate a fracture or re-open one. Whilst there is a general upward trend with depth for the pressures required for cracking, notably even at considerable depth, certain stress conditions require almost no resin pressure for crack initiation, and similarly at minimal depths certain stress conditions require considerable (20 MPa) pressures before cracks are initiated. From this it can be seen that the issue of resin pressures cannot be viewed from a ‘one-size-fits-all’ approach, but rather individual mines should consider their stress conditions and roof parameters before optimising the resin bolting parameters.

**Figure 2: Pressures required for crack initiation and crack re-opening in moderate strength (5 MPa tensile) strength rock for Australian real mine virgin stress conditions**

**REVIEW OF PAST RESEARCH**

Attempts to measure the effect of gloving on bond strength by partially over-coring gloved *in situ* bolts have shown a minimal decrease in load transfer as a result of gloving (Compton and Oyler 2005 and Craig 2012). Simulated gloving has been measured at up to an 85-90% decrease in load transfer (Mould *et al.*, 2004 and Pastars and MacGregor 2005). Different methods of measuring and expressing the severity of gloving have been used; most typically has been a general visual or external measurement of intact film longitudinally along the bolt. These range from arbitrary, qualitative assessments (Compton and Oyler 2005 and Villaescusa *et al.*, 2008), to quasi-quantitative measurements made by measuring the ‘length’ of gloving present (Campbell and Mould 2003). In reality, gloving interferes with the surface contact area between the resin/rock interface, and to truly quantify the amount of gloving experienced, the percentage of the surface area affected should be measured.

Recent investigations have focused on the issue of resin back-pressures and its effect on drill-hole fracturing and resin loss (Campoli *et al.*, 1999; Campbell and Mould 2003; Compton and Oyler 2005; Giraldo *et al.*, 2006; McTyer *et al.*, 2014 and Evans 2015). Reported factors that affect the magnitude of resin back-pressure include drill-hole annulus, resin viscosity, cartridge film thickness (Spearing *et al.*, 2011) as well as the resin length and the rockbolt insertion speed (both upward and rotational) (McTyer *et al.*, 2014). It was also suggested that the bar profile can have an impact on resin back-pressure (Evans 2015).

Several variations of testing methods have been used to investigate resin back-pressure. Rockbolt resin contains around 70% limestone particle fillers and commences reacting during bolt insertion. This makes it difficult to measure pressure using normal fluid pressure instruments. Direct pressure
measurements have been attempted by Compton and Oyler (2005) at the NIOSH testing mine in the USA using strain gauged instrumented steel pipe. Campbell and Mould (2003) reported attempts to measure fluid pressure during bolt and resin insertion into polycarbonate tube. All reported difficulty in inserting bolts into 'closed' pipes with bolts not reaching the back of the hole or polycarbonate pipes splitting during insertion. The installations do not compare to normal in situ rock installations. Since then, others have measured thrust force and attempted to calculate the pressure generated using the bolt or hole cross sectional area. (Giraldo et al., 2006 and Evans 2015). Evans’ (2015) calculations also went on to include the flow of the resin down the bolt correlated to bolt insertion speed. This research project will also seek to validate (or otherwise) the legitimacy of such calculations. A summary of previous research projects and the resultant resin back-pressure measurements is given by McTyer (2014), however importantly the measured pressures ranged from 4 to 45 MPa.

It is important to highlight that the measurements performed by researchers in the USA pushed the bolts through the resin without rotation as is standard practice in the US; rotation is only applied once the bolt is at the back of the borehole. Australian installation practice is to rotate the bolt at approx. 500 rpm during insertion up through the capsule. The cylinder pressure measurements by Compton and Oyler (2005) highlighted a sudden drop in pressure once rotation of the bolt commenced at the back of the hole. These findings raise questions as to the suitability of applying the US measurements/calculated pressures to the Australian system, as it is proposed that the absence of rotation exacerbates the pressure experienced in the resin.

To date, no researchers have directly measured the pressure generated during rockbolt and resin installation into rock.

**PROPOSED TESTING METHODOLOGY AND RESEARCH OBJECTIVES**

This research project attempts to provide an all-encompassing analysis of individual resin bolt system parameters (hole diameter, resin type, resin length, insertion rate and bolt profile) on key performance indicators (gloving, load transfer, rock fracture and resin loss/encapsulation) using consistent and reliable test methods, which simulate real world conditions as closely as possible; namely:

1) Measuring resin pressures of bolting systems with various parameter changes (setup shown in Figure 3).
   a. Preparing a rockbolt by cutting a slot along the axial length of the bolt, and a recess in the top of the bolt, to accept a small load cell.
   b. Placing the load cell on the tip of the bolt.
   c. Securing cable and data logger to the drive dolly.
   d. Measuring each borehole diameter using a micrometre at 100 mm intervals.
   e. Installing the rockbolt into a ~40 MPa reinforced concrete block using a hydraulic rig (identical to that used in the underground coal industry).
   f. Directly measuring the pressure generated in the resin, during bolt installation.
   g. Logging insertion speed, rotation speed and thrust pressure of the bolting rig.
   h. Measuring encapsulation.

Figure 3: Pictures of rockbolt and pressure cell setup: a) pressure cell countersunk in head of bolt and b) wiring at foot of bolt for attachment to logger
2) Overcoring and load testing of bolting systems with various parameter changes
   a. Measuring each borehole diameter using a micrometre at 100 mm intervals.
   b. Isolating drill rig thrust hydraulic circuit and installing a pressure gauge.
   c. Performing a ‘dry run’ installation with no resin to determine the thrust pressure required
to move the rig and rockbolt from its base position to the back of the hole.
   d. Recording drill rig thrust pressure during installation.
   e. Measuring encapsulation.
   f. Removing installed bolts and surrounding rock core from the strata by overcoring.
   g. Noting any resin migration from the borehole.
   h. Grouting the overcores into a 100 mm ID steel pipe using a TD80 cementitious grout
with a seven day UCS of 60 MPa. Once cured for seven days, cut into 100 mm lengths
using a band saw.
   i. Measuring push-out load of the bolt out of the rock core using an instrumented
compressive load test machine.
      i. Supporting the samples on a steel block containing a 34 mm diameter hole.
      ii. Positioning the rockbolt over the hole, allowing free movement whilst providing
support to the grout and rock column.
      iii. Arranging the specimen so that the applied load is oriented as if to push the
rockbolt ‘downwards’ with respect to the in-situ orientation of the overcore.
      iv. Noting the peak load and bond strength (defined as the point on the loading
graph at which the gradient drops below 20 kN/mm) of the resin for each
specimen (Reynolds, 2006). The testing setup is shown in
      v. Figure 4; the cross section of a sample is shown in
      vi. Figure 5 and an example of the push test result is shown in Figure 6.
   j. Quantitatively assessing gloving.
      i. Dismantling sections using an angle grinder.
      ii. Breaking away the rock and grout with a hammer to expose the resin column.
      iii. Removing any resin that was found on the resin/rock interface and measuring
its surface area.
      iv. Converting the measured surface area of film to a percentage (%) of the
theoretical surface area of the 100 mm borehole length.
Over-coring has been conducted on 27 bolts at 3 test locations at Chain Valley Colliery. Location 1 and 2 had very similar roof conditions which was comprised of 1500-1600mm of coal at the bottom, and 200-300 mm of tuff at the top of the bolting horizon. Location 3 cut horizon was slightly higher, meaning that only 700-800 mm of coal was present at the bottom, and 1000-1100 mm of tuff in the top of the bolting horizon. All the installations were completed as an outbye operation, and as such, the roof in the area of the test work had time to relax, possibly producing roof separations prior to testing. This is a further area of research that needs refinement, as ideally the bolts would be installed at the face.

A 100 mm/s rockbolt insertion rate was targeted for each installation. 27, 28 and 29 mm drill bits were used as part of this investigation.

Recording of Drill Rig Thrust Pressure

Further comments regarding the correlation between the rig thrust pressure and the actual pressure in the resin are detailed later in this paper, however preliminary results from the initial 27 bolts installed indicate that:

- Larger holes didn’t markedly reduce the thrust pressure required to install the bolt (see Figure 7)
- The thrust pressure required to install the bolt seemed to be less affected by fast insertion rate in large holes
- Resin type didn’t markedly reduce the thrust pressure required to install the bolt (see Figure 8)
- Viscosity of the resin didn’t markedly affect the thrust pressure required to install the bolt
- High speed insertion produces considerably higher thrust pressures than a controlled insertion rate.
- It takes approximately 40 bar (4) to raise the drill assembly, dolly and rockbolt at a controlled rate of 100 mm/s into a hole containing no resin (see Figure 9).

Considerable additional tests are required however in order to determine solid conclusions about the abovementioned parameters. Note: a) when looking at the following figures, the pressures quoted are direct hydraulic pressures for the rigs and have not been ‘converted’ to resin pressures, b) the graphs are for during both thrust-and-spin as well as spin at the back of the hole (with no thrust) and c) sharp
rises in pressure toward the end of the installation are due to the bolt contacting the back of the borehole.

Figure 7: Effect of hole size on rig thrust pressure

Figure 8: Effect of resin type on rig thrust pressure

Figure 9: Results of rig thrust pressures in ‘Empty Holes’
Push Tests

As almost all push tests failed at the resin to rock interface, no major conclusions can be made at this stage as to the quality of mixing or the effect of annulus size on the load transfer limit of the system, suffice to say that the ‘weakest-link’ of the load transfer system has seldom been found to be due to ineffective mixing or excessive annulus.

Gloving

Of the 17 bolts analysed, all but one were identified to be affected in some degree by gloving within the top 300 mm. Of the 51 x 100 mm samples inspected, 29 were identified as having some degree of gloving present.

The gloving and push test data was then collated across three horizons to measure the impact of gloving on peak load and bond strength. Specimens where rock had fallen away from the resin column during recovery of the over-core (which resulted in a grout on resin contact during preparation) were omitted from this investigation. The horizons investigated were three 100 mm sections of the Awaba Tuff, directly above the Chain Valley Coal Seam. The peak push out loads within the tuff were around 60-65 kN, and failure was always between the resin and rock interface. The results can be seen in Figure 10, showing a consistent trend of a reduction in both peak load and bond strength with an increase in gloving percentage. Gloving was seen to reduce the peak load of a 100 mm section by 4-6 kN (8-10% of peak load) per 10% of gloving affected surface area. At this stage of the testing there is significant scatter in this data, as these results contain numerous other variations in the rockbolting system parameters (such as resin type, borehole diameter and rockbolt type), however the aim of future testing will be to gather enough quality data to obtain statistically sound relationships between the various parameters.

![Figure 10: Peak load and bond strength vs gloving severity for horizons 1, 2 and 3](image)

Resin Loss

An assessment of resin loss was completed for each rockbolt installed in this testing program. Resin loss was calculated by comparing the theoretical encapsulation (using borehole micrometre measurements) to the measured encapsulation.

Thrust pressures were measured to range from 40 to 180 bar. No clear relationship was found when either the peak or average drill rig thrust pressure was collated with resin loss, as seen in
Preliminary testing with this setup has provided promising results; however further testing is required to draw any meaningful conclusions. Peak pressures were observed to range from 4-10 MPa, for ‘standard’ resin bolt parameters (1.8m long bolts, 21.7 mm diameter, 600-1000 mm long resin cartridges, installed into holes drilled with 28-30 mm drill bits). Further testing is planned using this method and variations will be used to assess various bolt/resin/borehole combinations to identify key parameters affecting the development of back-pressure. In addition, variations in the strength of the cement block material will be used to assess the impact of rock strength on the development of back-pressure. Utilising this testing method in an underground environment would be ideal, however investigations are continuing as to how to achieve this from an electrical approval perspective.

Preliminary results show that there is minimal correlation between the hydraulic rig thrust loads and the measured resin pressure using the load cells. Example results are shown in Figure 12.

Resin losses experienced in the concrete block were minimal (<10% variance from predicted encapsulation) for both 27 and 30 mm hole sizes.

CONCLUSIONS

Improved testing methodologies have been attempted in order to evaluate rockbolt performance. Measured performance parameters include not only regular load transfer capacity but also rock fracture, resin loss and gloving. The test methods have proven successful and a larger program is being developed. Some preliminary conclusions include;
• No marked reductions in drill rig thrust pressure was able to be achieved via variation of either hole size or resin type
• No correlation was able to be drawn between drill rig thrust pressure and resin loss
• Directly measured peak resin back pressures of between 4 – 10 MPa were recorded with 1.8m long Australian rock bolts and resin installed into a 40 MPa concrete block. These measurements are significantly lower than results from previous studies conducted in steel pipes.
• 16 of the 17 rockbolts recovered from underground were found to have experienced some degree of gloving within the top 300 mm of the bolt. On average, the severity of gloving was found to be relatively low.
• No correlation was able to be drawn between resin type, hole size, drill rig thrust pressure or bar profile and the severity of gloving
• A reduction in load transfer of 4-6 kN per 10% of gloved surface area was seen for push tested specimens. This equates to an 8-10% reduction of peak load per 10% of gloved surface area.

Whilst the inconclusive nature of the results so far limit the immediate benefit to the industry, they do give alternate and independent data by which to validate (or otherwise) previous claims made about various rockbolting parameters. Further research is required before the variables are able to be separated in order to decisively conclude various proposed theories.

Testing methodologies have been outlined which are suitable for the assessment of various resin anchored roof bolting parameters with the potential to provide optimisation of said parameters to suit the variety of conditions in underground coal mines.

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SPIN TO STALL BOLTING SYSTEMS – A REVIEW OF VALIDATION STANDARDS

David William Evans

ABSTRACT: Originating from South African practices, spin to stall bolting systems have seen an increasing focus within Australian coal mining operations over recent years. The primary driver for spin to stall applications is the goal of improved bolting cycle times, with a secondary benefit being simplified installation practices for roof bolting operators. These outcomes are seen as one part of a broader opportunity to improve overall underground mining efficiencies and extraction rates.

Spin to stall bolting methods represent a divergence from long standing resin installation practices, with the resin ‘hold’ or ‘curing’ time being eliminated from the installation cycle. The bolt can therefore be spun continuously under thrust through the length of the resin cartridge, with rotation continuing until the resin polymer matrix cures and hardens, the bolt torque drive releases, the nut is advanced on the thread and the bolt is ultimately pre-tensioned until rotational stall of the drill rig motor occurs. However, concerns remain that as the resin cures under polymerisation, significant loading is immediately placed onto the resin matrix before the polymer chains have fully formed, with the subsequent risk of damage to the interfaces between the bolt, resin and borehole – ultimately weakening bolt load transfer into the strata.

The divergence from standard resin bolting practices has also led to changes in product validation methods, with validation practices being created specifically for spin to stall testing. This paper provides a detailed analysis of the technical parameters that govern spin to stall installations, as well as an assessment of validation techniques, including the inherent difficulties associated with underground testing. The arising question is whether current validation practices actually address all aspects of bolt performance, with some areas for concern being potentially masked regarding bolt load transfer. Finally, this paper offers potential improvements for spin to stall validation methods, in an attempt to further progress this area of work within the industry.

INTRODUCTION

The primary function of roof bolting is to deliver one or more of four main support strategies – simple skin support, suspension of a thin roof layer from a massive bed, beam building of laminated strata and keying of highly fractured and blocky rock mass (Canbulat et al., 2005). Resin bolt performance, whether conducted as a standard installation, or as a spin to stall system, is ultimately about achieving the required geotechnical outcomes. Load transfer from the bolt, through the resin annulus and into the strata is a critical aspect of all four support strategies – providing transfer of forces from the mechanical support elements into the ground.

Traditional resin bolting practices employ a ‘hold’ time to permit polymerisation and curing of the resin. The hold time immediately follows mixing of the resin, and involves a period where there is no rotation or pre-tension applied to the bar. The bar is momentarily held in a static state – this enables the resin to cure around the profile of the bolt as well as adhere to the surface of the borehole. The resin curing time permits the development of polymer chains during chemical transformation of the two component resin cartridge. Longer polymer chains provide greater final strength of the resin matrix that has formed. The
length of the polymer chains is a function of the curing time – greater hold times permit longer polymer chains to form and subsequently produce stronger cured resins. There are two predominant installation practices that involve a hold time sequence. Australian roofbolting practices typically employ a combined spin and thrust through the resin, with additional rotation at the back of the hole for final mixing and then a final hold time to ensure correct polymerisation. After the hold time, the bolt is then pre-tensioned (Hillyer et al., 2013). This method produces greater mixing at the base of the resin column, and proportionately less mixing towards the top of the bolt.

A more conservative approach is typically taken in the USA coal industry, where the bolt is firstly thrust to the back of the hole without any rotation. Following full insertion, the bolt is then rotated to mix the resin. Following mixing, a hold time is finally employed to ensure correct polymerisation, before pre-tensioning of the bolt. This method ensures even mixing of the resin through the entire height of the resin column. Other variations of this method include slow rotation during bolt insertion, then a period of higher rotation to ensure correct mixing of the resin.

Spin to stall systems have a documented history of research dating back to the late 1990’s, with various practices that had originated from within South African coal mines (Canbulat et al., 2005). The main divergence from traditional installation methods is the removal of the hold time for spin to stall. However, multiple concerns were also raised during that same early assessment period regarding damage to the bonding between the bolt and the resin, including the need for increased roof bolt densities to compensate for weaker load transfers (Bigby et al., 2004 and Canbulat et al., 2005). Within the context of the developmental history of spin to stall bolting methods, the primary purpose of this current paper is to provide a review of the key parameters and methods of testing for spin to stall applications. While there is a growing body of work in the industry, a comprehensive standard remains to be more fully established.

MIXING AND LOADING SEQUENCES – SPIN TO STALL

Spin to stall practice places a demanding sequence of load immediately onto the resin during polymerisation and curing, within a comparatively rapid timeframe. For a dual speed resin, commonly used to provide pre-tension into the steel bolt element and generate compressive forces into the strata, a number of technical complexities arise. The ‘fast-set’ component of a dual speed resin cartridge is of course in the upper portion of the borehole. As the bolt rotates and thrusts, mixing from the bottom-up through the resin cartridge, it is the fast-set component that actually mixes last, but must react and harden first. Therefore, the fast-set component must react rapidly in order to take the sudden sequence of applied loads as the resin commences its transformation into a solid.

Firstly, the fast-set resin must adhere to and rapidly ‘grip’ the steel bar profile, to suddenly arrest rotation of the roof bolt. During release of the shear pin, rotational torque is transmitted directly onto the resin annulus at a value that is predetermined by the mechanical properties of the pin – as indicated in Figure 1a. This sudden arrest of rotational torque creates torsional shear forces through the entire resin annulus, including the associated interfaces with the bolt and borehole. Of course, lower shear pin release torques will produce a lower torsional force on the resin matrix, however, higher shear pin values are typically required to drive the bolt and mix the resin to the back of the hole.

For a 1.8 m long M24 roof bolt, within a 28mm borehole and utilising a 1000mm long resin cartridge with a 50:50 fast-set to slow-set ratio, the total resin volume can theoretically exceed 1700mm of encapsulation length – and the fast-set component is assumed to be at least 850mm long in the upper resin column. The applied release torque of the drive nut has been reported to be as high as 244 Nm (Emery et al., 2015). Based on an average roof bolt core diameter of 21.8 mm, a torsional shear force of 22,385 N is generated at the bolt-resin interface. For the 850 mm length of fast-set resin, the bolt-resin interface has a cylindrical surface area of approximately 58,221 mm$^2$ (excluding rib deformations). These results in a torsional shear stress applied to the resin interface of 0.384 MPa, assuming a uniform
gel time across the 850 mm fast-set column length. If the fast-set gel time is not uniform, for example, if applied only over 500 mm of resin, then the torsional shear stress becomes 0.654 MPa (or 6.67 kg/cm²).

Secondly, the fast-set resin must now take the pre-tension loads applied to the bolt – with the nut advancing on the thread and tensioning the bolt against the strata. This pre-tension force is also applied directly to the fast-set resin component, creating an axial shear force along the entire length of the hardening fast-set resin column. However, in combination with the axial force, a further torsional force is applied to the bolt associated with rotational tightening of the nut. So, the ‘pre-tension’ load is actually a combination of axial and torsional loads.

For an axial pre-tension load of 6 t (≈ 60 kN), a torque of approximately 200 Nm is applied during final tightening of the M24 nut. Applied across the 850 mm fast-set resin column, the torsional shear stress is 0.315 MPa and the applied axial shear stress is 1.031 MPa. The combined torsional and axial shear stresses induced during pre-tension totals 1.346 MPa (or 13.73 kg/cm²) at the resin-bolt interface. However, for an axial pre-tension load of 8 t (≈ 80 kN), a torque of approximately 270 Nm is applied during final tightening of the M24 nut. Across the 850mm fast-set column, this will induce a torsional shear stress of 0.425 MPa and an axial shear stress of 1.374 MPa – a combined shear stress of 1.800 MPa.

So, during polymerisation and curing of the resin, two significant loads are applied to the resin-bolt interface - an initial torsional shear stress that is induced during pin breakout of the nut and being potentially of the order of 0.654 MPa, followed by combined axial and torsional shear stresses that are induced during pre-tension and being potentially of the order of 3.059 MPa. Subsequently, it can be seen that the stresses induced during pre-tension are much more significant than the stresses induced during nut breakout by a factor of almost 4 to 5 times.

Figure 1: Forces induced on resin during spin to stall installations

The ‘slow-set’ component of the resin cartridge is positioned towards the bottom of the borehole. The slow-set component is designed to permit pre-tension and elongation of the roof bolt steel, before the slow-set resin finally cures and hardens, locking in the bottom half of the bolt profile onto the now
pre-tensioned bar. As the bolt rotates and thrusts, mixing from the bottom up through the resin cartridge, it is the slow-set resin that begins to mix first. The slow-set component of the resin will therefore see continued mixing for the entire duration, until the fast-set resin finally arrests rotation of the bolt. As a result, the slow-set resin risks ‘over-mixing’, where the polymer chains have moved beyond full formation and are subsequently broken down by continued rotation of the bolt. The result of over-mixing is solid resin that is weakened and brittle in appearance, dramatically reducing load transfer through to the strata.

**Resin mixing levels – spin to stall**

Various methods of bolt installation will produce different levels of mixing along the resin column. For combined spin and thrust through the resin, the amount of resin mixing will proportionately reduce through the height of the resin column. This can be represented by mapping out the cumulative number of revolutions of the bolt per unit height as the bolt advances through the resin column, as represented in Figure 2. This diagram assumes a constant rate of feed and rotation, then a period of rotation at the back of the hole, or rotational dwell, with the bar fully advanced in the borehole.

The timing sequence through spin to stall installations subsequently becomes critical. The bolt must thrust and mix at a rate where it is at the back of the borehole before the fast-set component begins to cure – otherwise the bolt will not fully insert and pre-tension. The speed of bolt insertion is further complicated by the potential for over-pressurisation of the fluid resin and the subsequent elevated risk of hydraulic fracture and loss of resin at the top of the bolting horizon (Evans 2015). There must also be a period of continued rotation on full insertion of the bolt to ensure mixing is adequate at the top of the borehole. So, the timing sequence for curing of the fast-set resin component must permit thrust through the resin column, as well as rotational dwell at the back of the hole, but avoid over mixing of the slow-set component – there is therefore a critical rotational dwell time, also shown in Figure 2.

![Figure 2: Cumulative mixing revolutions through the resin column](image)

For spin to stall applications, the rotational dwell time is actually governed by the fast-set resin curing time, and not by the mechanical cycle of the drill rig. Correspondingly, there is a greater degree of uncertainty in mixing due to variation in the curing time of the fast-set resin – given variances in strata lithology, ground temperatures, resin storage conditions and also equipment variables for thrust and rotation. Variation in the fast-set gel time will therefore cause variation in resin mixing times, ultimately
affecting the quality of the installed resin and the corresponding load transfer performance (Aziz et al., 2014).

As the bolt passes through the intermixing zone between the slow set and fast set resin, the lower region of the fast-set resin will begin to mix first – and this lower region will see the greatest number of mixing revolutions of the fast-set component. As a result, a critical length of fast set resin must solidify first in this same lower region to arrest the bolt rotation, but the upper portion of the fast set component will not see this same level of mixing – hence the inherent risk of un-mixing at the top of the bolt and subsequent variability in load transfer.

Current methods of underground testing

Complications arise when attempting to measure load transfer for spin to stall applications in the underground environment. For traditional resin installations, incorporating a hold time, 300mm Short Embedment Pull Testing (SEPT) is a well-known and commonly used method to determine load transfer. The 300 mm SEPT can utilise a specific mix and hold time to ensure integrity of the resin installation - and then provide a load transfer value over a known 300 mm horizon in the strata. However, for a spin to stall application, 300 mm of encapsulation is too short a length of resin to take the torsional and axial loads that are generated during polymerisation of the resin - a 300 mm resin encapsulation length will be destroyed by these forces and the test bolt will pull from the borehole.

The problem then becomes how to conduct a 300 mm SEPT for a spin to stall installation, given that encapsulation lengths greater than 300 mm are required to arrest rotation and pre-tension the bolt without causing significant resin damage. Previously reported testing methods for spin to stall installations (Altounyan et al., 2003 and Emery et al., 2015) included a split bolt format (SB-SEPT), where the upper 300 mm of the bolt is keyed and pinned and designed to release at low loads during underground pull testing. The top 300 mm of bolt is therefore seen as sacrificial, with plastic cartridge and unmixed resin potentially accumulating in this region. Just below the sacrificial region, the next 300 – 500 mm horizon is then pull tested.

However, it was further reported that the SB-SEPT method was modified to stop bolt rotation immediately after nut breakout (Emery et al., 2015). This means that the torsional loads from nut break out were applied to the resin, but the more significant pre-tension loads were not applied. As a result, the SB-SEPT test method is not a true test of the final installed load transfer associated with a spin to stall bolt, due to the absence of the pre-tension forces during resin polymerisation. The SB-SEPT method also reflects the general assumption that the top 300mm of the bolt is of limited value, presumably due to the effects of unmixed resin and gloving (Aziz et al., 2014). The subsequent risk is inconsistency in load transfer in the top 300mm of the bolt that cannot really be audited.

KEY PARAMETERS ASSOCIATED WITH SPIN TO STALL

It is important to fully define the parameters associated with a spin to stall installation, in order to correctly test the repeatability and quality of the bolt installation. While a number of these parameters are shared with traditional resin bolting methods that employ a hold time, there are additional factors that come into consideration for spin to stall applications. It is important that these parameters are properly defined, quantified and measured – the following is a summary of these parameters and their units of measurement.

Physical Geometry

- Borehole Dimensions - Diameter and Length (mm)
- Bolt Dimensions – Diameter, Length and Profile (mm)
- Resin Dimensions – Diameter, Total Length, Dual Speed Component Lengths (mm)
Resin Properties
- Gel times for fast-set and slow-set resin (sec)
- Punch shear strengths for fast-set and slow-set resin (MPa)

Bolt Rotation and Thrust Sequence
- Rotational speed (rpm)
- Rotational time to the back of hole (sec)
- Rotational time at the back of the hole (sec)
- Time to rotational arrest (nut breakout) (sec)
- Cumulative mixing revolutions (mapped per unit resin length) (rev)
- Time to pre-tension (sec)

Installation Loads
- Release torque at nut breakout (Nm)
- Pre-tension axial load (kN)
- Pre-tension torque (Nm)

Installed Resin Quality
- Load Transfer of Installed Bolt (kN/mm)
- Resin Failure Interface (bolt / resin / borehole)
- Bolt encapsulation length (mm)
- Resin mixing effectiveness (% mixed resin, mapped per unit bolt length)
- Gloving (% plastic content, mapped per unit bolt length)

Enhanced surface testing methods

In order to more fully measure the parameters associated with spin to stall applications, enhanced methodologies have been developed to increase the level of test data that is obtained for each installed test bolt. The enhanced test methods are based upon the utilisation of a pre-grouted steel pipe, with a bore hole pre-drilled to accommodate a spin to stall bolt installation, as shown in Figure 3. Instrumentation is incorporated to capture relevant installation data and the pre-grouted steel pipe later permits further test work to be conducted upon each installed bolt.

In preparation for the test, parameters are firstly recorded for the resin type, bolt type and the borehole dimensions - the bolt and resin are then positioned for installation into the grouted and pre-drilled pipe. As the bolt is installed under spin to stall conditions, data capture systems are utilised to record the installation parameters – specifically, the bolt displacement through the resin column and the rotational speed against time – enabling later graphical analysis of the full installation sequence, as shown in Figure 4. This data also provides a record of correct bolt installation, according to specified resin mixing and gel time sequences. The release torque of the nut is known from the shear pin type utilised and the final applied pre-tension is measured using a calibrated hydraulic load cell.

Following completion of the bolt installation, the grouted pipe can now be prepared for laboratory testing. The steel pipe is sectioned as indicated in Figure 5 - and un-needed areas of grout are broken away, producing two 300 mm long specimens for pull testing. The first specimen is from the top 300 mm of the bolt installation, representing the fast-set component of the resin at the top of the bolting horizon. The second 300mm specimen is taken from just below the mid portion of the bolt, representing the slow-set
component of the resin. Each test specimen has an embedment length of 300 mm, in common with SEPT methods utilised underground. Also, during sectioning of the grouted pipe, as the bolt tails are exposed, the resin in these regions can be assessed for quality of mix.

Figure 3: Spin to stall installation rig including instrumentation

Figure 4: Installation data for a spin to stall test – displacement and rotational speed

Figures 5: Sectioning of pre-grouted pipe containing an installed test bolt
Upon sectioning of the installation pipe, for both pull test specimens, a ‘tail’ of bolt is kept intact that now permits each test to be conducted as a pull test, rather than a push test. It is known that segmented laboratory push tests can provide contradictory results compared to pull testing methods (Aziz et al., 2006), and this dilemma is now avoided under the new technique. The two 300 mm specimens are then alternately tested under tension in a universal testing machine – measuring both the force and displacement associated with load transfer of the installed resin, as shown in Figure 6a.

Figures 6: Load transfer testing of 300 mm embedment specimens

There are two simultaneous modes utilised to measure and record displacement during the pull test. The first mode of displacement measurement is conducted utilising the cross-heads of the testing machine. This permits measurement of the yield of the steel bar relative to the load transfer that is being generated by the 300 mm sample. The second mode of displacement measurement is conducted with the use of a digital contact indicator positioned on the exposed top of the bolt, as shown in Figures 6b and 6c. This permits direct measurement of the displacement of the embedded bolt through the resin, enabling accurate measurement of load transfer stiffness.
Following completion of the load transfer tests, the two 300mm test specimens can now be further sectioned, as shown in Figure 7, to assess and analyse the resin and bolt interfaces – and whether the resin failure mode has occurred at the bolt surface, the borehole surface or due to shear of the resin. Also, upon visual analysis of the resin interfaces, the percentage of resin mixing effectiveness can be assessed (areas of over mixing and under-mixing), mapped out per unit length along the length of the pull test specimen. The percentage of plastic gloving present can also be assessed, mapped out per unit length along the length of the pull test specimen.

Figure 7: Sectioning and inspection of 300mm embedment specimens

This enhanced surface testing methodology provides a comprehensive set of data per test bolt installation. The validation methodology is fully measureable – installation parameters are captured, including the loads applied onto the resin during gel time. Load transfers are measurable via laboratory pull testing in a manner that is more in line with underground SEPT methods. Further to this, analysis of resin failure interfaces, mixing and gloving, are also enabled. Ultimately, the enhanced surface test methodology permits greater ability to benchmark and correlate laboratory based validation data with the bolt installation parameters of underground equipment.

PROPOSED UNDERGROUND TESTING METHODS

As previously discussed, a remaining area of difficulty is that of underground pull testing for spin to stall applications. While the split bolt (SB-SEPT) method provides a relatively low cost and convenient method, there are concerns regarding the incomplete loading of the test bolt during resin gel time, as well as the sacrificial 300mm length at the top of the test bolt representing potential un-mixing of the resin in this region.

An alternate method is proposed for underground validation testing of spin to stall bolting applications, with the intent to capture the correct sequence of installation loads and also provide a true 300 mm SEPT test at the top of the bolting horizon. As shown in Figure 8a, the new method is based upon bolt installation through a pipe and into an upper 300 mm section of the strata which is pre-drilled utilising a standard 28 mm bit. The pipe also has an internal diameter of 28 mm. The lower portion of the borehole is over-drilled to receive the larger outer diameter of the pipe. The pipe also has a torque arm to prevent rotation of the pipe, by way of securing the torque arm to an adjacent installed roof bolt. The pipe can also be coated with a zinc-rich coating to prevent spark generation as the roof bolt passes through, as
well as to slightly roughen the internal diameter of the pipe for resin adherence (Evans 2014). Grease is applied to the upper external surfaces of the pipe to prevent the adherence of any leaking resin between the borehole and the outer pipe wall. While the torque arm can be re-used between tests, the pipe is sacrificial and remains permanently in the ground along with the bolt.

Figures 8: SEPT apparatus for underground validation test work

Once the test equipment is fully prepared, the resin and bolt are positioned for installation under spin to stall conditions. The bolt rotates and is thrust through the resin, with mixing continuing at the back of the hole until gel time and rotational arrest of the bolt. At the point of release of the shear pin, the torsional shear forces are applied through to the internal surface of the pipe, as well as the upper 300 mm of the 28 mm borehole in the strata. The pipe is not able to rotate under these torsional forces due to the constraint of the torque arm.

After break out and advance of the nut, the bolt is now pre-tensioned. The pre-tension forces are applied axially and torsionally through the bolt and resin, and are subsequently constrained by the pipe, the torque arm and the upper 300 mm of the 28 mm borehole in the strata. In this regard, the pipe and upper 300mm borehole have acted in unison to constrain both the torsional and axial loads that are generated during the spin to stall installation.

Preparation is now conducted for the pull test - the torque arm is removed and a tripod pull test chair is placed over the plate with the support legs and pads resting against the strata as shown in Figure 8b. The coupler, pull rod and hollow bore hydraulic jacking cylinder are now positioned onto the chair and prepared for the test. If displacements are to be measured as a part of the pull test, they must be measured with respect to the bolt end and not the jack body, in order to capture bolt displacement and not jack movement. As the jack loads are brought onto the bolt during the pull test, the pipe will not transfer any axial tensile loads as there is no adherence to the strata walls. The axial test loads are now transmitted directly onto the upper 300 mm of the bolt in the strata, providing a true pull test of the upper 300mm bolting horizon.

The pipe method for conducting spin to stall SEPT is no doubt a more complex method and is not proposed for routine daily use. However, the pipe method can be used to provide true underground validation for the load transfer performance of a spin to stall bolt. Upon installation, the full column of resin, and in particular the upper fast-set component, is exposed to the true sequence of torsional and axial loads at gel time. The upper 300 mm of the bolt is installed directly to the strata at the bolting horizon, while the lower part of the bolt is essentially de-bonded from the strata by the pipe. In this way,
the bolt and resin column are installed under true spin to stall loading conditions, but the SEPT is conducted only on the upper 300 mm of the bolt.

CONCLUSIONS

Spin to stall resin installations represent a divergence from traditional resin bolting practices – predominantly due to the immediate sequence of torsional and axial loads that are applied to the resin as polymerisation and hardening occurs. Variations in rotational dwell time, as well as the initiation region of fast-set solidification, can risk under-mixing of resin at the top of the bolt and over-mixing of resin at the bottom of the bolt. Load transfer testing of spin to stall installations is also an area of difficulty, due to the need to replicate the torsional and axial installation loads across the entire bolt length, but to then provide an upper segment for pull testing that is independent from the lower strata. The enhanced surface testing methods described within this paper permit the replication of spin to stall bolting installations, with the installation parameters being defined and measured. Subsequent load transfer testing utilises laboratory methods that are more in line with traditional underground SEPT methods. An assessment of resin mixing effectiveness and plastic gloving is also incorporated within this test regime. Further to this, underground test methods are proposed that will enable SEPT of spin to stall resins - installed under true loading conditions and pull tested in the upper 300 mm of the bolt horizon.

This review is provided for the purpose of increasing industry awareness of the technical parameters associated with spin to stall resin bolting practices, as well as to identify improvement opportunities in related testing and validation methods. Ultimately, the geotechnical performance of the installed resin bolt is the primary required outcome – and this performance is predominantly a function of effective load transfer from the bolt, across the resin annulus and to the borehole. Effective load transfer remains an essential requirement for all resin bolting practices – and the associated methods of spin to stall validation should be improved to provide a more complete assessment of this critical aspect of strata control.

REFERENCES


A NEW CORROSION RESISTANT GROUND SUPPORT SYSTEM

Tom Meikle¹, John Bolton² and Bre-Anne Sainsbury³

ABSTRACT: The capacity of ground support components which have been affected by corrosion is reduced and may ultimately lead to failure of the component and the strata. In order to maintain an effective, long-term ground support system, significant campaigns of rehabilitation are often required in corrosion affected areas. The most common corrosion protection for steel ground support utilises sacrificial systems such as galvanising. Galvanising has previously been proven to be susceptible to some corrosion processes. Stainless steel is the most effective in resistance to corrosion, but can be cost prohibitive, and its mechanical properties often make it unsuited for use in ground support components.

Providing an outer protective plastic coating to bolts has proven to be an effective means of protecting the inner steel bar from corrosion. However, these support systems tend to be heavy, and require post point anchor cement grouting to provide full encapsulation. In comparison to a standard bolt/resin system, they are slow to install and expensive. These systems have also been shown to reduce overall load transfer performance of the bolting system.

In order to provide a higher level of corrosion protection whilst maintaining current installation practises and bolting cycle times, Minova has developed the Enduro™ steel ground support range. The Enduro™ range consists of standard Minova steel ground support components which have been treated with a unique coating process. The Enduro™ coating has been tested in the harshest of conditions, both underground and in controlled laboratory conditions. It has been proven to effectively resist or eliminate the formation of corrosion, even in the most aggressive environments. This paper outlines the laboratory and underground corrosion performance testing carried out on Enduro™ ground support products.

INTRODUCTION

Traditionally, the most common form of corrosion protection in steel ground support consists of a sacrificial protective coating such as galvanising. Such coatings have proven to be ineffective in extreme high and low pH conditions with corrosion of the support system commonly encountered. There is also evidence that common forms of galvanising may actually increase the rate of corrosion in certain pH environments. Figure 1 shows typical corrosion and failure on a standard re-bar roof bolt.

Figure 1: Corroded and failed roof bolt

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In order to provide additional protection, Double Corrosion Protected systems were introduced. These systems, although effective in providing additional corrosion protection, have proven to be expensive, complex to manufacture, bulky and difficult to handle. They are normally slow to install which leads to a reduction in development rates. In addition to these challenges, Clarke and Sieders (2014) identified that the plastic layer between the steel bolt and anchor may impact the axial load stiffness of the ground support.

Due to these limitations in the double corrosion protection systems, Minova developed the Enduro™ range of steel ground support products. The Enduro™ bolt, whether it be a solid bolt or friction bolt, is installed like any other common bolting system. It is no heavier than a standard bolt and requires no additional training or special equipment to be installed.

**ENDURO COATING**

There are two components to the Enduro™ coating:

1. The Enduro™ or base coat which covers the entire surface area of the bolt. The Enduro™ coating is a unique and protected Minova application to ground support components.
2. An optional top coat applied to either the entire surface area or selected sections (i.e. the exposed tail of bolt).

The Enduro™ coat is applied using the Cathodic Dip Coating (CDC) process. In this process the system applies a Direct Current (DC) charge to the component, which is immersed in a bath of oppositely charged coating particles. The particles are drawn to the component surface and are deposited forming an even, continuous film over the surface (including every crevice) until the coating reaches the desired thickness, typically 100 µm.

An optional top coat is applied using a Thermoplastic Powder Coating (TPC) process and is suitable for most metals that can withstand 180°C oven temperatures that are required for curing the powder. The thickness of TPC is typically 150 µm -250 µm. The top coat can be used to provide extra confidence in the protection of the steel, particularly where physical damage through extreme handling may be encountered. It is also effective in UV protection to the Enduro™ coat should the tail be exposed.

The thickness of either the Enduro™ or top coat can be increased, or the coating process repeated to provide even greater confidence in the corrosion protection.

**LABORATORY TESTING AND RESULTS**

In order to validate the performance of the Enduro™ product, a series of controlled laboratory tests have been completed. A summary is provided below.

**Corrosion Resistance Acetic Acid Salt Spray (AASS)**

The corrosion resistance performance of the Enduro™ product in ASS has been observed over 1000 hours and compared with a traditional galvanised bolt. A total of three bolts were tested that included a ‘standard’ galvanised bolt, a bolt with an Enduro™ base coat and a bolt with an Enduro™ base coat and a topcoat.

The salt spray chamber used to perform the tests is shown in Figure 2.

Test conditions were pH 3.1 – 3.3 @ +35°C. The test results for each of the three coating types are provided in Table 1 and Figure 3.
Despite galvanised bolts being used extensively to protect against corrosion in saline conditions, laboratory testing conducted herein presents obvious signs of corrosion. Better corrosion resistance has been observed (when compared with the galvanised bolt) by the two Enduro™ variant coated bolts. The product with both base and topcoat provided the best resistance to corrosion under saline conditions.

**Acid bath immersion**

The acid resistance of the Enduro™ product in a low acid environment has been observed and compared with a traditional bolt materials. A total of three bolts were tested that included a ‘standard’ hot-dip galvanised bolt, a bolt with an Enduro™ base coat and a bolt developed from raw steel.

A straight forward test procedure was used whereby each of the products were placed in an acidic solution (dilute sulphuric acid with a pH of 1.69) for a period of 50 days. The visual appearance was observed and recorded at 30 minute, 5 day, 21 day and 50 day increments for each of the bolt types.

### Table 1: Acetic Acid Salt Spray Results

<table>
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<th>518</th>
<th>722</th>
<th>1012</th>
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<tr>
<td><strong>Description</strong></td>
<td>Corrosion observations after elapsed hours</td>
<td>Corrosion observations after elapsed hours</td>
<td>Corrosion observations after elapsed hours</td>
<td>Corrosion observations after elapsed hours</td>
</tr>
<tr>
<td><strong>Required Specifications</strong></td>
<td>No visible corrosion observed for acceptable performance result</td>
<td>No visible corrosion observed for acceptable performance result</td>
<td>No visible corrosion observed for acceptable performance result</td>
<td>No visible corrosion observed for acceptable performance result</td>
</tr>
<tr>
<td><strong>Galvanised Bolt</strong></td>
<td>Severe white rust, some red rust</td>
<td>Severe white rust, some red rust</td>
<td>Severe white &amp; red rust</td>
<td>Severe white &amp; red rust</td>
</tr>
<tr>
<td><strong>Observed Result</strong></td>
<td>Fail</td>
<td>Fail</td>
<td>Fail</td>
<td>Fail</td>
</tr>
<tr>
<td><strong>Enduro™ Bolt (base coat only)</strong></td>
<td>No visible corrosion</td>
<td>No visible corrosion</td>
<td>Minor red rust spots</td>
<td>Minor red rust spots</td>
</tr>
<tr>
<td><strong>Observed Result</strong></td>
<td>Pass</td>
<td>Pass</td>
<td>Fail</td>
<td>Fail</td>
</tr>
<tr>
<td><strong>Enduro™ Bolt (base coat and topcoat)</strong></td>
<td>No visible corrosion</td>
<td>No visible corrosion</td>
<td>No visible corrosion</td>
<td>No visible corrosion</td>
</tr>
<tr>
<td><strong>Observed Result</strong></td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
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</table>

![Figure 2: Salt spray chamber used for Enduro™ product testing](image)

![Table1: Acetic Acid Salt Spray Results](table)

![Acid bath immersion](image)
The acid bath and pH measuring instrument is shown in Figure 4.

![Figure 4: Acid bath and pH monitoring instrument used during Enduro™ product testing](image)

Immediately upon immersion, the hot-dip galvanized bolt (grey colour – Figure 4a) commenced ‘fizzing’ (reacting). The raw steel bolt (charcoal colour - Figure 4b) commenced reacting after approximately 30 minutes. Over a period of testing there was no apparent reaction with the black Enduro™ base coated bolt (Figure 4c).

After 5 days of immersion, a pH of 2.25 was measured. Sheets of corroded material can be seen on the galvanized and raw steel bolt. There was no corrosion observed on the Enduro™ bolt (Figure 5 left). After 21 days, additional corrosion was observed on the raw steel and galvanised bolts (Figure 5 right).
After 50 days (Figure 6) both the raw steel and galvanised bolts have corroded significantly with the nut on the galvanised bolt completely corroded. There were still no obvious signs of corrosion on the Enduro™ bolt.

From the observations it is clear that the galvanised and the raw steel bolt were severely corroded after 50 days of immersion. The Enduro™ bolt still looked 'intact' with no apparent corrosion evident. The dissolution pattern of the raw and galvanized bolts appears different:

- the raw bolt appears to be dissolving uniformly with the nut showing severe corrosion.
- the galvanized bolt shows severe pitting and complete dissolution of the nut.

In both cases, the threaded areas of the raw steel and galvanized bolt area has been significantly reduced in diameter. The Enduro™ nut and thread is observed to be unaffected showing no signs of corrosion (Figure 6).

**Accelerated corrosion tests**

Monash University has conducted accelerated corrosion tests to provide a rapid comparison of the corrosion resistance of the Enduro™ product to other products in the marketplace and determine corrosion expected from 2 years to 100 years (specifically thickness loss data in mm/y).

To complete the tests, a test solution of 3.5 wt% NaCl, was prepared by dissolving 35 g of NaCl in 1 L of distilled water. This solution has a normal pH of approximately 7.8. For the development of other pH solutions, 0.1 N solutions of HCl or NaOH were added to achieve pH environments of 2, 7 and 9 for
testing. The accelerated corrosion testing procedure used included taking potentiodynamic polarization measurements using a typical three-electrode electrochemical cell that was connected with a Bio-Logic potentiostat. Prior to each corrosion test the Open Circuit Potential (OCP) was measured to confirm stability over a period of 1000 seconds. A total of five bolt materials were tested that included; Enduro™ with base coat, black (e.g. ungalvanised), hot dip galvanisation, Thermally Diffused Galvanisation (TDG) and stainless steel. Results for each of the products at pH 2 are presented in Figure 7.

![Figure 7: Average calculated corrosion rate of Enduro™ and comparison samples in 3.5% NaCl solution, pH 2](image)

The Enduro™ product specimens provided the best observed corrosion resistance in acidic conditions (3.5% NaCl at pH 2) followed by stainless steel and then the other products tested. It can be observed that HDG and TDG coatings have extremely low resistance to corrosion from low pH solutions.

The corrosion performance of the Enduro™ product (basecoat only) over a range of pH conditions is provided in Figure 8 for the purposes of estimating service life.

![Figure 8: Enduro Corrosion (mm/yr) calculated at various pH levels](image)

FIELD TESTING AND RESULTS

A series of underground tests on the Enduro™ product were carried out to determine the durability of the bolt when exposed to highly acidic mine water. A summary of the in situ trials are provided below.

Mine A

Six sections of Enduro™ bolts (Type A) and six sections of Hot Dip Galvanised bolts (Type B) were selected for use in this trial. Each section was 600 mm in length. The bolts were placed underground in free running/draining acidic water as shown in Figure 9. The pH was measured once a week and is also provided in Figure 9.
Figure 9: Mine A Bolt in situ test conditions

After 30 days, a section of each bolt type were removed from the acidic water and the excess build up was removed and the level of corrosion observed (Figure 10). It can be seen, that under the highly acidic mine water conditions at Mine A, the Enduro™ bolt outperforms the hot dip galvanised bolt in resisting the effects of the corrosive mine water.

Figure 10: Mine A after 30 days (left) bolts observed underground (right) excess build up removed from bolts

Figure 11 presents a solid Enduro™ bolt that has been installed for six weeks at Mine A. At this point discoloration is observed but no corrosion is evident on the bolt. The galvanized mesh surrounding the Enduro™ bolt is eight weeks old.

Figure 11: In situ observations of the performance of an Enduro™ bolt in comparison to galvanized mesh in acidic mine groundwater conditions
Mine B

At Mine B, *in situ* trial sites for the Enduro™ product were selected based on a significant observed degradation in the existing galvanised split-set support which had been installed approximately three years previously and was re-supported. Figure 12a shows a heavily corroded split set and plate that has been installed for approximately two weeks. Visual observations regarding the corrosion of the bolts have been made in addition to corrosion assessments of the inside of the bolts using a hand-held wand camera (Figure 12b). The performance of the Enduro™ product was measured (along with what is called red bolt) over a two month period whereby exposure of the bolts was maximised by placing them on the floor under dripping mine water (Figure 13). In order to replicate mining conditions the bolts were extensively scratched and scoured on the inside and outside. The extent of corrosion on the bolts was recorded every month. The red-bolt and the black Enduro™ bolt showed minimal evidence of corrosion over a two month period.

![Figure 12: Observed corrosion of existing galvanized ground support through surface and penetrating visual means at Mine B](image)

![Figure 13: Observed corrosion of Enduro™ (black) and (red-bolt) at Mine B](image)

Underground installation at Mine C

Minova was approached regarding the support system currently being installed in Mine C’s surface to underground declines. Mine C were having issues complying with the geotechnical design criteria to fully encapsulate the bolt prior to the application of shotcrete. At the time Minova got involved, this issue was severely hindering development rates and had led to a substantial amount of re-support having to be carried out.

Minova proposed using the Enduro™ solid bolt along with our standard resin anchor to overcome this issue. Having proved the durability of the Enduro™ bolt, the system was adopted. Mine C was able to accept a less than 100% encapsulation of the bolt due to its effective resistance to corrosion. Implementing this system also eliminated manual handling issues and exposure to cement dust encountered while using the previous post grouting system. As a result of the application of the Enduro™ product, advance rates increased significantly and no further re-work was required.
CONCLUSIONS

Field and Laboratory testing of Enduro™ rockbolts and ground support components have shown significant reductions in rates of corrosion compared to other traditional corrosion resistant steel ground support systems. Additionally, Enduro™ bolt installation offers significant efficiency benefits over other rock bolts in projects designed for a life of 100 years. It is believed that project designers and Geotechnical Engineers now have a viable new ground support system to consider where long service life is required and / or extreme corrosion potential exists.

ACKNOWLEDGEMENTS

The Authors would like to acknowledge the following people who have contributed significantly to the development and testing of the Enduro Rockbolt range including; Steve Burgess, Evan McKay, Professor Raman Singh, David Cuello and Peter Keall.

REFERENCE

MECHANICAL PROPERTIES OF GROUTS AT VARIOUS CURING TIMES

Ali Mirza, Naj Aziz, Wang Ye and Jan Nemcik

ABSTRACT: The uniaxial compressive strength, elastic modulus and creep of two commonly used grout products in Australian coal mining industry, Jennmar Bottom-Up 100 (BU100) and Orica Stratabinder HS were studied. A 50 mm cube mould was used to cast samples. The Uniaxial compressive strength of the samples with curing time, ranging from 1 to 28 days was determined, using an Avery compression testing machine. Secant, tangent and 50 kN range elastic modulus of grout products were investigated under cyclic loading at a 1 mm/min loading rate. Both strain gauged and non-strain gauged samples were tested for determination of the elastic modulus in compression. Strain gauged samples were prepared to evaluate creep values of BU 100 and Stratabinder HS under 100 kN compression load for 15 min. It was found that Stratabinder HS had higher uniaxial compressive strength and elastic modulus after a day of curing in comparison to BU 100. Stratabinder HS showed lower creep values by 0.04% when compared with BU 100.

INTRODUCTION

Application of cable bolts as a common secondary support system to reinforce incompetent strata in Australian underground coal mines goes back to the 1970s. Cable bolts, unlike ordinary rebar bolts, are mostly installed using cementitious grouts. Recent developments in cable bolt design have increased the crucial role of grout products to act as a stable interface between the cable bolt and surrounding rocks with the aim of keeping the underground openings safe and stable for a long period of time.

Cable bolt failures have been observed in various Australian underground coal mines with no evidence of strand rupture. These failures can be attributed to installation practices and premature failure of grout. Encapsulated cable bolts provide an effective support system over a very large span for blocks and wedges formed in roofs or walls of excavations. The cable bolt reinforcement system consists of four main components as:

- Strands
- Barrel and wedge
- Grout
- The rock

The above mentioned components interact with each other to transfer the load of loose strata to deeper and more stable rock structures. Any defects or possible overload in components may deteriorate the whole support system, leading to failure. Therefore, it is of utmost importance to determine the mechanical properties of components, delineating the maximum load at which the whole cable bolt support system can stay in one piece without failure. A series of experimental studies was carried out to investigate the Uniaxial Compressive Strength (UCS), Elastic modulus (E) and creep of two commonly used grout products in the Australian coal mining industry.

There are two main research studies in literature concerning the mechanical properties of resin and grout. The first one was carried out by Aziz et al., (2013a, 2013b, 2014a) with the aim of establishing a general practice standard for determination of mechanical properties of resin used mostly for rebar bolt encapsulation. The study included determination of UCS, E value in compression, shear strength and rheological properties. Mechanical properties were examined at the University of Wollongong laboratory.

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in relation to resin sample shape, size, height to width or diameter ratio, resin type, resin age and cure time. The following main conclusions were reported from this investigation:

- 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 to cylindrical specimens at various curing time.
- Similar to UCS values, the average shear strength increased with larger sample 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., (2014b) through the Australian Coal Association Research Scheme (ACARP) organisation. The second research study in this area was performed by Hagan and Chen (2015) at the School of Mines, University of New South Wales. The Stratabinder product was selected as a grout material due to its low shrinkage and low viscosity. The UCS values of cube and cylindrical samples at different water to cement ratio were investigated, and it was found that:

- Cube samples provided higher UCS values when compared to cylindrical specimens, and.
- Strength of the Stratabinder material varied with water to cement ratio, showing a reduction trend with increase in the water quantity.

SAMPLE PREPARATION AND EXPERIMENTAL PROCEDURE

Two types of grout product, Jennmar Bottom-Up 100 (BU100) and Orica Stratabinder HS were selected to prepare samples. Samples were cast using the 50 mm cube mould with mixing ratios of 5 and 7 litres/bag by weight of grout to water, respectively. During sample preparation mild vibration was applied to the mould to release any entrapped air. Samples were then left undisturbed to cure at the room temperature for 1, 7, 14 and 28 days. Also Shown in Figure 1 is the effective process of mixing grouts for cube casting. A Universal 50 t Instron and a 90 t Avery conventional compression testing machine were used in the various tests.

EXPERIMENTAL RESULTS AND DISCUSSION

Uniaxial compressive strength (UCS)

More than 18 compression tests were carried out on prepared samples at 1, 7 and 28 days curing time. Some tests were repeated to ensure accuracy of the collected data. Figure 2 shows one day old, 50 mm cube samples of BU 100 and Stratabinder HS grouts.

The average UCS values of BU 100 and Stratabinder HS samples at different curing times are presented in Figure 4. It appears that the UCS strength of BU 100 and Stratabinder HS increased with longer sample curing time from 1 to 28 days. In general, Stratabinder HS samples failed at higher compression loads than those of BU 100. The exception was one day cured specimens whereby BU 100 performed better. All samples failed in compression tests along shear planes as a result of combined axial compression and transverse extension (Figure 5).
Figure 1: [left] a close view of 50 mm cube mould [right] smooth and consistent slurry

Figure 2: Prepared samples at 1 day curing time [left] BU 100 [right] Stratabinder HS

The UCS values of one day old BU 100 and Stratabinder HS samples are shown in Figure 3. The obtained UCS values varied between 45.46 to 54.18 MPa and 40.09 to 43.2 MPa for BU 100 and Stratabinder HS respectively.

Figure 3: UCS values at one day curing time of (A) BU 100 and (B) Stratabinder HS
Modulus of elasticity in compression (E)

In the determination of E values, samples were subjected to a cyclic loading program for three repetitive cycles, at loading rate of 1 mm/min (Figure 6). The maximum load at each cycle was limited to about 80% of the failure load. After three loading cycles, the prescribed load increased monotonically until failure. Both strain gauged and non-strain gauged samples were tested at 14 days curing time. Three methods (secant, tangent and 50 kN loading range) were used to calculate E values as described by Aziz et al., (2014a and b).

Figure 7 presents elastic modulus where strain values were obtained using the Instron machine for BU 100 and Stratabinder HS products. The E value for both products was found to range between 2.63 to 4.33 GPa. Stratabinder HS showed higher E values when compared with BU 100. The maximum
difference between E values of BU 100 and Stratabinder HS products is 0.98 GPa in tangent elastic modulus while minimum difference is 0.19 GPa in the 50 kN range method.

![Figure 6: Typical loading program on Stratabinder HS sample](image)

Table 1 compares the elastic modulus in compression where strain values were recorded through strain gauges and Instron machine. It is observed that E values obtained from strain gauges are higher than those of the Instron machine. This is because the strain gauge measures displacement in the middle of the sample unlike the Instron machine, which records total sample height compression. Similar results were reported by Aziz et al., (2014a) for resin products.

**Creep property**

Creep is a measure to represent deformation under the influence of mechanical stress which is less than the yield stress. Strain gauged samples of BU 100 and Stratabinder HS with curing time of 42 days were subjected to a constant compression load of 100 kN for 15 min. The loading rate from 0 to 100 kN was set at 2 kN/sec. The creep was calculated as the percentage difference between strain values at the end of the test and the onset of 100 kN loading.

Figure 8 shows measured strain values under the prescribed constant load of 100 kN compression load for BU 100 and Stratabinder HS samples. Three tests were carried out on each type, giving maximum strain values of 0.27 % and 0.22 % for BU 100 and Stratabinder HS respectively. Comparison between average creep values of BU 100 and Stratabinder HS is presented in Figure 9. Stratabinder HS offers higher resistance against constant load of 100 kN rather than BU 100. However, the difference between creep values of BU 100 and Stratabinder HS is 0.04% which is not significant.

![Figure 7: Comparison between elastic modulus of BU 100 and Stratabinder HS products](image)
Table 1 Comparison between elastic modulus determined from strain gauges and the Instron machine

<table>
<thead>
<tr>
<th></th>
<th>BU 100 - Gauge</th>
<th>BU 100 - machine</th>
<th>Stratabinder HS - gauge</th>
<th>Stratabinder HS - machine</th>
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</thead>
<tbody>
<tr>
<td>Secant (GPa)</td>
<td>18.21</td>
<td>2.62</td>
<td>13.9</td>
<td>3.33</td>
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<tr>
<td>Tangent (GPa)</td>
<td>26.84</td>
<td>3.33</td>
<td>26.84</td>
<td>4.31</td>
</tr>
</tbody>
</table>

Figure 8: Creep test under 100 kN compression load for (A) BU 100 and (B) Stratabinder HS

Figure 9: Comparison between creep values of BU 100 and Stratabinder HS

CONCLUSION

The experimental study found 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. It was also observed that 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.

REFERENCES


MODELLING OF DYNAMIC FRACTURE PROPAGATION IN COAL PILLARS USING FLAC 2D

Gaetano Venticinque and Jan Nemcik

ABSTRACT: For many years the empirical prediction of pillar stability has been the dominant method for pillar design. Now, with the advanced numerical modelling of dynamic fracture propagation, it is possible to study the correlation between the empirical pillar load estimates and the actual pillar fracture mechanics. An upgraded version to previously developed constitutive FISH subroutine in FLAC 2D driving the User-Defined-Model which simulates compressive failure behaviour of coal pillars through the development of dynamic fracture propagation. Special insight on peak strength and post failure behaviour is presented through analysis of fracture development as a function of the pillar width to height ratio. Results derived as part of this study show that the numerical fracture propagation in coal pillar produces similar results to the classical empirical estimations of pillar peak loads. The classical progressive shear failure in pillar ribs was observed in the models depicting the probable rib failure mechanisms. Such model is well suited towards providing better understanding of fracture behaviour in rock or coal mass to improve safety when mining within or around complex geological structures.

INTRODUCTION

A new method of modelling dynamic fracture propagation in rock has been used to simulate coal pillar failure modes. The new FISH code within the FLAC2D software was written to enable dynamic fracture to propagate through the rock, coal or similar materials. Several pillar geometries confined between competent roof and floor strata with various width to height (W/H) ratios of mined seam were modelled. The coal pillars were gradually loaded to failure and the progressive fracture propagation within the coal was simulated. For each pillar geometry, the modelled pillar strength was then compared against current popular empirical equation predictions.

THE FRACTURE MODEL

The dynamic fracture propagation model is a constitutive FISH based UDM in Itasca (2007) FLAC 2D geotechnical software; offering independent simulation of rock fracture initiation and propagation behaviour. This model has been previously verified for isotropic rocks where it was proved to offer realistic simulation of the brittle fracture and post failure response of rock in FLAC. In its current form, the model accommodates all three combinations of fractures: Mode I tensile, Mixed Mode I-II tensile shear and Mode III pure shear. This was verified through simulated application over the entire brittle-to-ductile transitional failure range of rock Venticinque (2013).

Simulation of Fracture Propagation in Coal Pillars

A new subroutine within the FLAC model was constructed to simulate coal pillar stress-strain response over the range of pre-and-post failure loading. This was performed for a number of pillar width to height (W/H) ratios listed in Table 1. Application of Venticinque’s (2013) updated fracture model code was employed during each simulation to provide insight into key mechanisms of pillar failure achieved through the generation of real time dynamic fractures.

The typical geometry of modelled pillars is shown in Figure 1. Load was applied to the model through an application of steady strain loading at the grid surface boundary. The roof and floor were assumed competent whilst the properties of the coal seam were weaker in comparison. Due to the geological
complexity of coal mass, the detailed structures such as the cleat and the bedding planes were not modelled. The properties of modelled rock layers are listed in Table 2.

Table 1: (W/H) Ratios of Simulated Coal Pillars

<table>
<thead>
<tr>
<th>(W/H) Ratio</th>
<th>Width (m)</th>
<th>Height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td>2:1</td>
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<tr>
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<td>10.5</td>
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<td>4:1</td>
<td>14.0</td>
<td>3.5</td>
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<td>5:1</td>
<td>17.5</td>
<td>3.5</td>
</tr>
<tr>
<td>8:1</td>
<td>28.0</td>
<td>3.5</td>
</tr>
</tbody>
</table>

The typical geometry of modelled pillars is shown in Figure 1. Load was applied to the model through an application of steady strain loading at the grid surface boundary. The roof and floor were assumed competent whilst the properties of the coal seam were weaker in comparison. Due to the geological complexity of coal mass, the detailed structures such as the cleat and the bedding planes were not modelled. The properties of modelled rock layers are listed in Table 2.

FLAC GRID 400x60

![FLAC grid model geometry](image)

Figure 1: FLAC coal pillar model geometry

Table 2: Pillar model layer properties

<table>
<thead>
<tr>
<th>Coal Layer Properties</th>
<th>Competent Roof and Floor Layer Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Modulus (GPa)</td>
<td>Bulk Modulus (GPa)</td>
</tr>
<tr>
<td>Shear Modulus (GPa)</td>
<td>Shear Modulus (GPa)</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>Density (kg/m³)</td>
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<tr>
<td>Tension (MPa)</td>
<td>2700</td>
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<td>Internal Friction (°)</td>
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<td>Dilation (°)</td>
<td>6</td>
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<tr>
<td>Density (kg/m³)</td>
<td>1400</td>
</tr>
</tbody>
</table>

Simulation Results

The average pillar load vs % of strain of all modelled pillars with various W/H ratios are plotted in Figure 2.

Modelled pillars are further investigated across Figures 4 to 9 which detail the progression of fracturing and key failure mechanisms over the entire (W/H) range.

At the early stages of pillar loading the fracture initiation and propagation begins at the pillar sides for all pillar sizes. Simulated fractures were predominately of a shear nature with dip angles approximately 60°
agreeing with the theoretical description of π/4 + Φ/2 for uniaxial loaded samples, (Jaeger and Cook 1971). Some minor tensile fractures also developed but did not influence the results greatly. As pillar loading increased, shear fractures were observed to work their way deeper into the pillars, Figure 3.

Figure 2: Average load vs strain for simulated coal pillar W/H ratios

Figure 3: Progression of simulated fractures within modelled pillars

The narrow pillar (W/H = 1) failed at the maximum load of 6.3 MPa at the strain of 0.13%. The abrupt collapse of the pillar occurred early with no post failure strength and several shear fractures spanning between the roof and the floor as shown in Figure 4.
The pillar of (W/H = 2) performed in a similar manner to the (W/H = 1) with slightly larger strain at failure as plotted previous in Figure 2. The coal pillar of (W/H = 3) developed early shear fractures at the pillar ribs with the fracture zone progressing deeper into the pillar as the vertical load increased. The modelled peak pillar load reached 7.8 MPa while the pillar post failure strength gradually reduced to low values indicating pillar inability to support additional load at greater strain. The load history versus the pillar strain and the fracture locations are shown in Figure 5.

It can be seen in Figure 5 that the post failure capacity of the pillar (W/H = 3) can be either zero or close to zero. It is noted that the waviness of the stress strain curve is related to subsequent individual fracture propagation.
For the (W/H = 4) the increasing vertical load caused the progressive rib shear failure to continue deeper into the pillar sides reaching the pillar centre at the strain of 0.85%. The maximum pillar load prior to total coal failure reached 11.1 MPa followed by a minor reduction in post failure pillar bearing capacity between 9.5 to 10 MPa, shown in Figure 6.

![Figure 6: Average load vs strain and failure mode for pillar (W/H = 4)](image)

The coal pillar with (W/H = 5) geometry followed the trend of the progressive pillar shear failure shown in Figure 6 with the maximum pillar capacity of 37.3 MPa. The pillar post failure strength stabilised at approximately 34 MPa indicating that even under higher strain the pillar will always carry some residual load bearing capacity. The wider coal pillar model of (W/H = 8) followed a similar progressive failure trend, Figure 8; however, the ability to accept the increasing load continued with the increase of strain. This behaviour conforms with anticipated mechanics of pillar coal systems described by Gale (1999).

![Figure 7: Average load vs strain and failure mode for pillar (W/H = 5)](image)
Through assessing the progressive failure within the coal pillars at various (W/H) ratios, several important observations became apparent and are summarised below:

- **(W/H = 1)**, Peak load of 6.3 MPa at strain of 0.13%.
  - Note: The dynamic fracture propagation in the small pillar can be abrupt with no post failure strength. The peak load that the pillar can sustain is close to the *in situ* coal strength.

- **(W/H = 3)**, Peak Load of 7.8 MPa at strain of 0.23%.
  - Note: The pillar exhibits fast but more gradual fracture propagation towards the centre. Very small post failure strength was observed.

- **(W/H = 4)**, Peak load of 11.1 MPa at strain of 0.95%.
  - Note: The pillar exhibits gradual fracture propagation towards the centre. The stress-strain curve resembles an elastic-plastic behaviour.

- **(W/H = 5)**, Peak load of 37.3 MPa at strain of 3.93%.
  - Note: The pillar gradually yields towards the centre. The stress-strain shows strain softening behaviour with the maximum load peaking at a relatively high strain.

- **(W/H = 8)**, No peak load, load continues to increase with strain.
  - Note: Gradual yielding from the pillar edges eventually stops at 8 m from the pillar ribs.

### COMPARISON OF MODELLED PILLAR STRENGTH WITH EMPIRICAL FORMULAS

The ultimate compressive load that the modelled coal pillars were able to carry were compared against empirically derived methods. In this analysis three well known empirical methods by Salamon-Munro (1967), Holland and Gaddy (1964) and Bieniawski (1992) were selected for comparison against simulated pillar strengths. The respective equations along with calculated values are given in Table 3 and 4 with results plotted in Figure 9.
Table 3: Empirical Coal Pillar Strength Formulas

<table>
<thead>
<tr>
<th>Empirical Method</th>
<th>Formula</th>
<th>Key</th>
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<tr>
<td>Salamon and Munroe (1967)</td>
<td>$\sigma_p = 7.2 \times \frac{W^{0.46}}{H^{0.66}}$</td>
<td>$W =$ Pillar Width $H =$ Pillar Height</td>
</tr>
<tr>
<td>Holland and Gaddy (1964)</td>
<td>$\sigma_p = k \sqrt{\frac{W}{H}}$</td>
<td>$k =$ Constant Relating to UCS</td>
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<tr>
<td>Bieniawski (1992)</td>
<td>$\sigma_p = \sigma_1 (0.64 + 0.36 \times \frac{W}{H})$</td>
<td>$\sigma_p =$ Peak Pillar Strength $\sigma_1 =$ UCS</td>
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Table 4: Calculated strengths for coal pillar W/H ratios simulated

<table>
<thead>
<tr>
<th>(W/H) Ratio</th>
<th>Peak Pillar Average Strengths (MPa)</th>
<th>Salamon and Munroe</th>
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Figure 9 – Comparing peak pillar strength vs W/H ratios for simulated coal pillars

Discrepancies between empirical methods and deviation of calculated strengths particularly for (W/H > 4) reflects a general limitation of empirical formula to be over conservative at (W/H > 5) as shown in Figure 9. Empirical approaches are henceforth recognised to be applicable only for conditions similar to those under which they were developed (Jaiswal 2009). Given the conditions simulated the results indicate that the correlations between the empirical pillar strength predictions and the modelled pillar
peak loads for various pillar geometries (W/H) ratios is good. Noted ability of numerical methods is highlighted where they are better capable of performing analysis of pillars with (W/H > 4) without being overconservative.

CONCLUSION

Overall, good comparison between the empirical data and the numerical model indicate that that the fracture mechanics within the model are realistic and can be used to study the sudden or progressive rib failure mechanisms that exist underground. In the empirical prediction of pillar failure the coal has to be the weakest component of the system. This does not apply to the numerical models where fractures can propagate in any location whether in the roof, coal seam or the floor strata. This makes the model more desirable to study the influence of the roof, coal or floor strata failure on the maximum pillar load. In many cases the weak contacts between the coal and the rock strata can significantly reduce the pillar stability Gale (1999). Likewise, excessive roof and/or floor failure in adjacent mine roadways can also influence the pillar strength by reducing the ability of the stiffer rock strata to provide lateral confinement to the coal seam. This numerical model has no problem in modelling such cases. Further research is planned to study how the coal cleat and laminations affect the fracture propagation in the coal mass. This study would enable more accurate predictions of the pillar strength. Through continued validation and additional application towards other complex problems Venticinque’s (2013) model is envisaged to offer better, safer, more stable and efficient results to the mining and wider geotechnical industries.

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USE OF 3D LASER SCANNER TECHNOLOGY TO MONITOR COAL PILLAR DEFORMATION

Radovan Kukutsch, Vlastimil Kajzar, Petr Waclawik and Jan Nemcik

ABSTRACT: Although the room and pillar mining method is world-known and widely used, in terms of the Czech coal mines located in the Upper Silesian Coal Basin it is still in the testing phase. Unfavourable mining, geotechnical conditions at large depths and the ban by Czech mining authorities prevented this method from being used on rock bolt reinforcement without other roof support. Typically, large amounts of unexploited coal reserves are left in the shaft protective pillars. This coal can be mined if strata subsidence is minimised. Due to its low subsidence characteristics the room and pillar mining method without pillar extraction has been trialled at the CSM Coal Mine at the end of 2014. During the pillar development phase complex geotechnical monitoring was undertaken including the frequent scanning of pillar movement using 3D laser scanning technology. The laser scanner enabled complex capture of the entire space around the monitored pillars during the period of pillar formation and afterwards. The time-lapse scanning method measured changes in the mine roadway surface profiles including pillar displacements, roof movements, floor heave and other dynamic phenomena. The time-lapse scanning indicated variable pillar rib movement ranging from a few cm to a maximum of 50 cm with an average of approximately 25 to 30 cm. The scans indicated that the bottom of the seam displaced more than the top of the rib side due to large floor heave. The weak floor consisting of siltstone and coal beds experienced large floor heave however, due to floor brushing no reliable floor displacements are available. In contrast to the large movements in the rib and the floor, the strong roof strata did not show any significant movements. The purpose of this work is to highlight the importance of terrestrial laser scanning as an essential engineering design tool to evaluate the displacements and deformations of mine excavations at large depths. The 3D scanning results gave relevant information about displacements and deformations that occurred at the tested site and thereby helped to improve safety underground.

INTRODUCTION

Laser scanning has started to be a widely applied technology in a variety of industries. These scanning systems excel in their ability to provide contactless determination of spatial coordinates of any object such as buildings, interior space, terrain and other structures. To survey the entire area of interest it is usually necessary to take scans from several locations that are automatically stitched together. The method can be used with exceptional speed, accuracy, comprehensiveness and safety. The scanned objects are visualised in the form of a point cloud, which can be subsequently used for a wide variety of analytical tasks, and also to generate 3-dimensional models of these objects.

This technology can be widely used in mines for specific tasks such as monitoring of strata conditions when developing mine excavations, assessing the long term stability of underground workings, measuring convergence profiles, investigating surface movement and enabling volumetric calculations. Based on the intensity of the reflected laser signal, it is also possible to quantify the material type, from the resulting point cloud. The method can also be used in design work connected with CAD / GIS tools.

The primary objective of laser scanning underground is a complete capture of the scanned area geometry at the time of scanning. From these results, it is possible to determine the shape and size of scanned mine workings, the position of mine supports and other parameters (Kajzar et al., 2015). Beyond these basic tasks it is also possible to use the spatio-temporal analysis, combined with the convergence measurements to compare the time related changes that may include: the size variation

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and stability of rock cavities and to identify the place and magnitude of coal pillar deformation and rib conditions or collapse of pillar corners. These tasks require repeated measurements to resolve any geometrical changes between the scans (Kukutsch et al., 2015). In general any number of future scans can be performed and processed to compare with the existing scan results taken in the same area.

MINING CONDITIONS IN THE SCANNED AREA

The geological setting in the area of shaft protective pillar ČSM-North Mine is quite complex. The targeted coal seam No. 30 used for the trials, is at the depth of approximately 700 m to 900 m below the surface. Above the coal seam there is a 300 m thick complex carboniferous rock mass with the overlying tertiary sedimentary rock strata 400 m to 600 m thick with approximately 20 m thick quaternary soil overburden. The strata dip oriented in a north-east direction ranges from 8° to 17°. Occasionally the dip of a coal seam can reach up to 20°.

Within the proposed mining area the thickness of seam No. 30 is very variable. In places the seam splits to several separate coal seam layers. Interchangeable layers of sandstone, siltstone and coal seams are present. Seam No. 30 occurs separately only in the south-east part of the shaft protective pillar with thickness that varies from 1.8 m up to 2.2 m. The seams n.n. (untitled seam), No. 31 and No. 32 merge with seam No. 30 towards the north-west. This substantial and complex coal seam (consisting of seams 30 + n.n. + 31 + 32) has the thickness of up to 5.2m in the northwest part of the protective pillar area.

In the monitored pillar area (pillars V1 and V2) in the panel V trial area the 3 m thick seam consists of the coal seams 30 and n.n. The stratigraphy of the ground above the coal seam No. 30 are shown in Figure 1.

![Figure 1: 3D model of monitored pillars in panel V (locality A). Coal seams – blue, siltstones – green, sandstones - yellow](image)

There are several faults of regional importance in the area of the ČSM-North shaft protective pillar (see Fig. 2). There is a wide tectonic zone of the Albrechtice Fault with total throw up to 420 m located in the west area. The dip of this fault ranges 60° to 65° towards the West. In the northern area “Fault A” is present with a throw of up to 100 m and the dip of 60° towards the North. “Fault B” in the south part of area has a throw of around 10 m with the dip ranging 55° to 70° towards the South.
The significant regional tectonic fault zone “Eastern Thrust” divides the area of the protective pillar into two separate blocks with different geotechnical conditions. According to the existing knowledge the Eastern Thrust has a very small dip ranging from 10° to 35°. The Eastern Thrust strike is generally in the NE-SW direction with the dip towards the NW. The vertical displacement fluctuates at around 5 m mark, but the range of horizontal displacements is usually much greater and can exceed tens to hundreds of meters. Characteristic changes in the Eastern Thrust dip with depth have been observed and may be correlated with the transition to interlayer slips. The experience shows that these thrust fault features have a significant effect on the geotechnical conditions within the rock mass.

Inside the protective pillar area surrounded by faults of regional importance, the rock mass is typically disturbed by a system of small so-called seam faults. The uplift on these seam faults is mostly greater than 0.1 m but typically do not exceed 1 m.

Figure 2: Tectonic situation and position of monitored pillars in panel V (locality A) and panel II (locality B)

MONITORING – DESCRIPTION

Stress monitoring and the deformation state of the rock mass is an essential requirement for the design of a safe and successful room and pillar method that can be applied in the Czech part of the Upper Silesian Coal Basin (USCB). The room and pillar mining method is usually designed on the basis of experience and practices that are observed under different geological conditions and depths. The geology in the area and depth of cover indicate that the empirical methods of calculating the pillar loads may not be appropriate and could be unreliable. No experience of the room and pillar method exists within the USCB area therefore the pillar monitoring had to be used to measure the capacity and the deformation characteristics of the coal pillars.

An extensive monitoring system was installed to measure the load profile across the coal pillar and the deformation characteristics of the pillars during mining. The monitoring was performed in two coal pillars within the panel V. The pillars diamond in shape and with slightly irregular sides were approximately 860 m² and 1200 m² in size and 3.5 m high.

In the context of stress and deformation monitoring the following parameters were measured:

- deformation of rock overlaying the room and pillar roadways,
- pre-mining stress and stress changes in rock and coal during mining,
- deformation of coal pillars,
load on the installed cable bolts,
roadway convergence,
repetitive 3D laser scanning of pillars and mine roadways.

To monitor roof deformation, fourteen pairs of 5-level multipoint extensometers monitored roof displacements and eleven strain gauged rockbolts were installed at various locations. Two 3-dimensional CCBO stress overcoring cells were used to measure the pre-mining stress in the area and eight 3-dimensional CCBM stress change monitoring cells were installed to measure stress changes during mining. Four 1-dimensional hydraulic stress monitoring cells were installed at various depths into each pillar to measure vertical stress, four 5-level multipoint rib extensometers measured displacements of all sides within each monitored pillar, seven hydraulic dynamometer load cells measured the cable bolt loads installed at the roadway intersections and the roof and rib convergence was measured at key locations. The coal rib displacements together with the convergence measurements, changes in vertical pillar loads and the monthly 3D laser scanning of the overall roadway displacements (roof, rib and floor heave) provided data to evaluate pillar stability. Large seismology and seismo-acoustic monitoring was also undertaken to supplement the data.

**3D LASER SCANNER MONITORING – DESCRIPTION**

For monitoring of spatio-temporal changes to the coal pillar V2 rib profile compact pulsed terrestrial laser scanner Leica ScanStation C10 was used. It is a device with a long range laser beam (up to 300 m) which provides measurements with accuracy in space (6 mm / 100 m), length (4 mm / 100 m) and angle (±60 micro-radian) and high scanning speeds (up to 50,000 points / s) (Leica – C10 Data Sheet).

Since this is a site with a potentially explosive atmosphere and the surveying equipment does not correspond to the safety criteria, monitoring was performed in all cases with a special permit and the use of a comprehensive gas monitoring system.

Up-to-date the laser scanning has been carried out six times within the approximate time interval of 5-6 weeks (see Table 1). The initial scan has provided geometry of the newly developed coal pillar V2 within the roadways V300501 and V3006 (see Fig. 3). When the pillar sides were completed the subsequent scans established the pillar boundaries within the corridors V3005 and V300502 (see Fig. 4). Following scans measured any subsequent changes due to strata displacements. Scanning was performed with a resolution of 1 cm / 10 m, while approximately 14.5 million spatial points from each scanning position were obtained.

A local coordinate system is required to tie all surveyed results together. For the scanning instrument to locate itself, several permanent (overlaying) target locations need to be established and marked (usually on the steel bolt plates that do not move). During each survey several magnetic targets (minimum of three) are placed onto these locations with some overlap between the targets that were used in the adjacent scans (common targets). The instrument can be set up anywhere at a desirable location where the targets are visible. The instrument then locates itself automatically and tie all the surveyed points together.

**Table 1: Laser scanner surveys up to date.**

<table>
<thead>
<tr>
<th>Color</th>
<th>Scan No.</th>
<th>Date</th>
<th>Number of positions</th>
</tr>
</thead>
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<tr>
<td>■</td>
<td>1</td>
<td>2015, February 10</td>
<td>6</td>
</tr>
<tr>
<td>■</td>
<td>2</td>
<td>2015, March 17</td>
<td>8</td>
</tr>
<tr>
<td>■</td>
<td>3</td>
<td>2015, April 21</td>
<td>6</td>
</tr>
<tr>
<td>■</td>
<td>4</td>
<td>2015, June 4</td>
<td>7</td>
</tr>
<tr>
<td>■</td>
<td>5</td>
<td>2015, July 21</td>
<td>8</td>
</tr>
<tr>
<td>■</td>
<td>6</td>
<td>2015, September 3</td>
<td>8</td>
</tr>
</tbody>
</table>
Figure 3: Part of the pillars V2 at the intersection of corridors V 300501 aV3006 (1st scan)

Figure 4: Shape of the pillars V2 - top view (6th scan)

DYNAMICS OF COAL PILLAR DISPLACEMENT

Raw data were processed using Leica Cyclone, Trimble RealWorks and CloudCompare software. After each survey it was necessary to merge the data (taken on the day) together (registration process). The second process consists of interconnecting the current survey with the previous surveys into a new single point cloud.

From the resultant data clouds the relatively accurate locations of the scanned surfaces are calculated and displayed. The individual profiles of each scan can be plotted as shown in Figure 5 showing the roadway cross-sections. The readily available software can be used to study individual displacements at the points of interest. Further visual display options of the calculated net displacements between each scan include the colour plot of the surface displacements as shown in Figure 7. These tools enable further study of the overall displacements and substrate conditions of each scanned surface as they develop in time. These surveys clearly indicate the overall pillar displacements, roof conditions and floor heave as they develop during mining and afterwards.
From the comparison in Table 2 it is clear that:

- Varying degrees of rib displacement rate change in time (Figure 6) occur in the whole profile of the pillar. Significant coal pillar deformation, accompanied by a continuous coal rib spall into the roadway area were encountered in the roadways V3006, V300501 and V300502. Rib displacements of up to 50 cm were measured in the coal pillars. These results correlate with the outcomes and findings from the convergence and extensometer measurements. Within the belt roadway V3006 the pillar deformation was much lower.

- The reinforcement performance and functionality can also be assessed from the scanned roof or rib bolt movements and the overall roof convergence. As in the case of rib displacements it is possible to detect significant changes in floor deformation. However these changes are difficult to assess as they are affected by a combination of different factors such as floor heave, floor brushing and moving mining equipment. The 3D laser scanning technology also enables the long term monitoring of strata conditions and thus proved to be beneficial for this project where pillar creep may occur.

<table>
<thead>
<tr>
<th>Cut No.</th>
<th>Width 1st scan (m)</th>
<th>Width 6th scan (m)</th>
<th>Difference (m)</th>
<th>Change (%)</th>
<th>Height 1st scan (m)</th>
<th>Height 6th scan (m)</th>
<th>Difference (m)</th>
<th>Change (%)</th>
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<tr>
<td>C1</td>
<td>7.31</td>
<td>6.74</td>
<td>0.58</td>
<td>7.90</td>
<td>2.88</td>
<td>2.56</td>
<td>0.33</td>
<td>11.32</td>
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<td>C2</td>
<td>5.76</td>
<td>5.05</td>
<td>0.72</td>
<td>12.46</td>
<td>3.30</td>
<td>2.98</td>
<td>0.32</td>
<td>9.67</td>
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<tr>
<td>C3</td>
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<td>7.39</td>
<td>0.23</td>
<td>3.05</td>
<td>4.37</td>
<td>3.73</td>
<td>0.64</td>
<td>8.34</td>
</tr>
<tr>
<td>C4</td>
<td>6.56</td>
<td>6.10</td>
<td>0.46</td>
<td>7.00</td>
<td>3.74</td>
<td>3.48</td>
<td>0.26</td>
<td>3.93</td>
</tr>
</tbody>
</table>

In Figure 7, parts A and B show the dynamics of the rib in a timeframe of approximately 7 months (6 scans) during and after the V2 pillar development. The part A shows the rib displacements ranging 15-20 cm in five weeks after the pillar was formed. In the following five weeks the rib displacements increased by 5-7 cm to approximately 20 to 27cm. These were the maximum changes measured in the timeframe of seven months. It appears that the bottom of the rib side displaced more than the top. This phenomenon may be related to the large floor heave, which partially affected the bottom half of the pillar.
where the displacements of more than 40 cm can be seen. Another reason may be the uneven cut in the V300501 roadway (see Fig. 4). Thus this cut may have a negative effect on the adjacent rib movement. This can be investigated further by plotting horizontal movement together with the rib geometry in a plan view.

Figure 6: Dynamic of coal pillar / decrease in displacement/time rate over time

Part A of Figure 7 corresponds to the location of the pillar extensometer VeH2 (Figure 7, middle of cuts in part A). The extensometry results and the colours obtained from Figure 7 are compared in Table 3. It should be noted that the colour range in Figure 7 and Table 3 can be changed to enable more accurate comparison for interpretation. Figure 7 clearly shows that the rib displacement rate is decreasing over time as shown in Figure 6. Further measurements will be conducted in the future to establish whether the pillar will stabilise. Compared to Part B (Figure 7 and Table 3) there are no significant rib displacements in Part A. Much larger displacements can be clearly seen in Part B.

This method of displacement reading by colour range shows the strength of the laser scanning. The time dependent displacements in space can be easily obtained and interpreted providing a quick and safe method to enable geotechnical assessment of strata conditions underground.

<table>
<thead>
<tr>
<th></th>
<th>1st month</th>
<th>2nd month</th>
<th>3rd month</th>
<th>4th month</th>
<th>5th month</th>
<th>6th month</th>
<th>7th month</th>
<th>8th month</th>
</tr>
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<td>VeH2 rib displacement (mm)</td>
<td>72.3</td>
<td>116.3</td>
<td>144.7</td>
<td>170.1</td>
<td>186.7</td>
<td>201.7</td>
<td>211.9</td>
<td>217.6</td>
</tr>
<tr>
<td>Increase in VeH2 displacement (mm)</td>
<td>72.3</td>
<td>44.0</td>
<td>28.4</td>
<td>25.4</td>
<td>16.6</td>
<td>15.0</td>
<td>10.2</td>
<td>5.7</td>
</tr>
<tr>
<td>Corresponding color scale (used in Fig 7 Part A)</td>
<td>not measured</td>
<td>Blue Range 50-150mm</td>
<td>Blue Range 50-150mm</td>
<td>Blue Range 50-150mm</td>
<td>Green Range 150-250mm</td>
<td>Green Range 150-250mm</td>
<td>Green Range 150-250mm</td>
<td>to be measured</td>
</tr>
<tr>
<td>Corresponding color scale (used in Fig 7 Part B)</td>
<td>not measured</td>
<td>Blue Range 50-150mm</td>
<td>Green Range 150-250mm</td>
<td>Green Range 150-250mm</td>
<td>Yellow Range 250-300mm</td>
<td>Orange Range 300-400mm</td>
<td>Red Range 400-500mm</td>
<td>to be measured</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The room and pillar method has been trialled in the shaft protective pillar at the CSM Mine within the USCB. A comprehensive coal pillar monitoring was essential as this was the first application of the room and pillar mining method in the Czech Republic at great depth. Two coal pillars located in the seam No. 30 were intensively monitored to ensure stability of the panel and safe mining procedures. Several monitoring instruments were used to measure stresses and displacements in this trial. One of these instruments was the 3D laser scanner to measure strata surface displacements during and after mining took place.

The measurements indicate that the 3D laser scanner is a comprehensive tool enabling both numerically and graphically the express the dynamic changes taking place in the mine roadways. The laser scan results from several subsequent surveys indicted significant coal rib displacements taking place shortly after mining started. The measured rib displacements varied and ranged from 15 to 20 cm when mining took place increasing after 7 months to 20-27 cm at pillar mid height. Post-mining rib displacements were also measured. The results indicate that the lower coal rib suffered greater deformation mainly due to large floor heave that occurred along all sections of mine roadways. In some places the lateral displacements measured mostly at the lower rib exceeded 50 cm. As expected, the roof displacements were negligible and are not discussed here. These scanned data were directly compared with the extensometry results.
Graphic display of the measured changes that occurred, enabled easy and immediate evaluation of the strata conditions up to the measured date. Based on these data the level of risk can be established whether to abandon the mining area in case of severe pillar deformation.

Application of the 3D laser technology to measure strata displacements in underground mines is becoming increasingly popular due to low cost of measurements its ease of use and data quality it produces. Graphic outputs of the measurements enable quick assessments of the situation at hand. Over the coming years this method will inevitably become an essential part of the safe and economic monitoring system used on regular basis to provide quick and reliable information on strata conditions in real time

ACKNOWLEDGEMENTS

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REFERENCES

THE EFFECT OF ENERGY TRANSMISSION ON MINE COAL PILLARS

Faham Tahmasebinia¹, Ismet Canbulat¹, Chengguo Zhang¹, Serkan Saydam¹ and Luming Shen²

ABSTRACT: Random kinetic energy induced from strain energy stored in mining structures can distribute the stresses in rock masses. This physical transformation from potential to kinetic energy can lead to a severe coal burst which can be highly damaging. An efficient tool that can evaluate this stress distribution can play an important role in the design and planning of coal pillars and mine layouts. This paper presents a novel three-dimensional finite element modelling methodology (3D FEM) that has been developed to determine the structural response of a pillar subjected to kinetic energy release. This methodology can be used to determine the areas where a pillar is susceptible to violent, uncontrolled failure as well as to study the structural responses of a coal pillar. As part of the study a parametric study of combination of softening parameters in both coal and coal/rock interface was conducted to determine critical regions in the pillars that may lead to a better design strategy in coal burst prone mines.

INTRODUCTION

Galvin (2015) stated that, the terms most commonly used to describe dynamic energy releases in underground coal mining are pressure (or coal) bumps and pressure (or coal) bursts. Both terms refer to dynamic energy events associated with stress levels in the rock mass (or coal). However, the commonly accepted difference between a pressure bump and a pressure burst relates to the magnitude and hence, the consequence. A pressure bump is a dynamic release of energy within the rock mass (or coal) 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 excavations. A pressure bump is also sometimes referred to as a bounce. A pressure burst is a pressure bump that actually causes consequent dynamic rock/coal failure in the vicinity of a mine opening, resulting in high velocity expulsion of this broken/failed material (or shakedown) into the mine excavation. The energy levels, and hence velocities involved can cause significant damage to, or destruction of conventional installed ground support elements such as bolts and mesh.

In metalliferous mining, a strain burst is usually referred to as a seismic event caused by a failure of a localised, relatively small volume of highly stressed rock in the immediate vicinity of an excavation. A rock burst, on the other hand, is a higher-energy event that can range up to magnitude 5 on the Richter scale. Most pressure bursts associated with coal mining would be classified as strain bursts in the hard rock mining sector (Galvin 2015). Coal burst has been recognised as one of the most catastrophic failures associated with coal mining, which can lead to injuries and fatalities of miners as well as significant production losses (Kusznir and Farmer 1983; Brauner 1994; Iannacchione and Zelanko 1995; Potvin 2009; Mark 2014 and Galvin 2015). Coal bursts are usually classified as a natural phenomenon directly attributable to the coal becoming over stressed. A number of techniques and methods have been developed in the past to attempt to determine the potential and critical zones for rock bursts in underground mines. Some of the techniques have been derived from the balance of energy around excavations, including a combination of strain energy, kinetic energy, and potential energy. Cook (1963) developed an Energy Release Rate (ERR) concept which has become one of the most popular techniques among the methods currently available (Cook 1976; Linkov 1994; Wang and Park 2001 and Wattimena et al., 2012).

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MODELLING STRATEGY

As part of this study, a bord-and-pillar model to evaluate energy transmission in different strata layers due to different properties has been developed. A Mohr-Coulomb (MC) material that presents a constant strength after failure, and a Mohr-Coulomb strain-softening material (Wang and Park 2001; Islam et al., 2009; Sirait et al., 2013; Mortazavi and Alavi 2013; Nie et al., 2014 and Poeck et al., 2015) that can reach the peak strength and then decrease to a residual strength have been considered. It is suggested that the outcomes of the numerical modelling study together with the combination with other analytical techniques can be used to estimate both in situ stress as well as mining induced stress, where it may result in identifying the coal burst prone areas in a mine site. Another aspect of this study is that it takes into account the influence of the third dimension which can play a key role in interpretation of the result. Developing 3-dimensional Finite Element (FE) Models using dynamic solver (ABAQUS/Explicit), which is a convergence free solver, is one of the major advantages of the current simulations in comparison with the former simulations. Moreover, Poeck et al., (2015) emphasised the advantages of the three dimensional FE modeling in comparison to 2-D models when considering the correlation of energy release values.

Table 1 lists the basic material properties used for overburden and coal material properties.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m$^3$)</th>
<th>Young's Modulus (Pa)</th>
<th>Poisson Ratio</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (Pa)</th>
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<tr>
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<td>2350</td>
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<tr>
<td>Coal</td>
<td>1313</td>
<td>3e9</td>
<td>0.2</td>
<td>23</td>
<td>1.69e6</td>
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</table>

Table 2 lists the changes in cohesion, friction and dilation angles applied to the strain-softening material with associated levels of strain (Poeck et al., 2015). A bord-and-pillar panel layout in conjunction with different material properties, joint properties, and loading conditions were undertaken by Poeck et al., (2015). In order to comprehensively extend the Poeck et al.,’s model (2015), a 3D pillar model that considers the different joint properties has been developed in this study (Figure 1). Consideration was also given to defining a joint interface between the coal and overburden rock.

<table>
<thead>
<tr>
<th>Strain</th>
<th>Cohesion (Pa)</th>
<th>Strain</th>
<th>Friction angle (deg)</th>
<th>Strain</th>
<th>Dilation angle (deg)</th>
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<td>0.00000</td>
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<td></td>
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</tr>
</tbody>
</table>

Figure 1: Illustration of a typical single pillar model
Based on the study by Poeck et al., (2015), the three variations of joint properties included in the study were fixed (or tie condition) where there is no slip between the rock and coal interface. The Coal-rock interface Slip (CS) is presented by Coulomb-slip parameters, and Continuous Yielding (CY) is presented by displacement softening parameters. Table 3 lists the parameters applied to each of the constitutive joint models and Figure 2 shows the stress/strain behaviour of the MC and CY joints used in the coal/rock interface.

**Table 3: Joint properties used for the coal/rock interface (Poeck et al., 2015)**

<table>
<thead>
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<th></th>
<th>Coulomb Slip</th>
<th>Continuously Yielding</th>
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<td>Shear Stiffness (Pa)</td>
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<td>50.0e9</td>
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<tr>
<td>Normal Stiffness (Pa)</td>
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</tr>
<tr>
<td>Cohesion (Pa)</td>
<td>0.0</td>
<td>------</td>
</tr>
<tr>
<td>Dilation angle (deg)</td>
<td>0.0</td>
<td>------</td>
</tr>
<tr>
<td>Tensile Strength (Pa)</td>
<td>0.0</td>
<td>------</td>
</tr>
</tbody>
</table>

**Figure 2: Stress/Strain behaviours used in coal/rock interface (after Poeck et al., 2015)**

**NUMERICAL MODELLING SIMULATIONS**

The numerical modelling layout presented in this paper was conducted using the commercial software package ABAQUS. All materials, including the rock and coal were modelled with the eight-node linear brick element (C3D8R) available in the ABAQUS library. Element C3D8R relies on reduced integration and hourglass control, and its meshing is carried out with the structured technique available in ABAQUS. The solution to the nonlinear problem was sought using the explicit dynamic analysis procedure available in ABAQUS (Tahmasebinia et al., 2012). This approach is an improvement to an implicit formulation as it can handle the convergence problems encountered with nonlinear analyses of composite members efficiently when dealing with complex joint conditions.

In the previous studies, it was noted that ABAQUS/Standard could not ensure convergence of all simulations included in their realisations at high levels of deformation, despite the FE solution relied on the RIKS (which is a static solver) method based on an arc-length control procedure (Tahmasebinia et al., 2013). The explicit dynamic analysis adopted in this study uses an explicit integration rule, where the
equation of motion of the model is integrated in time using the explicit central-difference rule (ABAQUS User’s Manual 2008). To perform quasi-static analyses with this approach, it is appropriate to artificially increase the mass of the model in order to keep its kinematic energy minor. This is achieved by using the FIXED MASS SCALING option available in ABAQUS, which requires utilisation of the minimum time increment used in the analysis based on which ABAQUS/Explicit determines the mass scaling factors adopted in the calculations.

DISCUSSION OF THE RESULTS

Different material properties and joint properties were simulated and tested. The results indicated that the softening behaviour in the Mohr–Coulomb has no significant influence on the absorption of strain energy. The major sources of the strain energy might be concerned with the rock or coal ejected when the coal burst takes place, and that kinetic energy of that material after the burst equals all of that strain energy minus the work that has to be done to create a crack (or series of cracks) to detach it from the surrounding rock or coal. However, when joint properties are considered, those that are continuously yielding would be presented best by CY and can play a key role on changing level of strain energy. With the tie condition (i.e., where there is no slip between the engaged surfaces), the energy released from the rock mass would be limited. This phenomenon indicates how ductility between the interfaces can change the failure mode as it can determine the levels of kinetic and strain energies. As an example, the Energy Release Rate (ERR) has been presented in different conditions as presented in Figure 3.

![Figure 3: Energy Released Rate, Mohr = Mohr-Coulomb rock mass, CS: Coulomb slip interface, CY: Continuously Yielding interface)](image)

The same properties are used to compare in-plane horizontal stress distributions throughout the critical sections where it is situated near the edges in different material and joint properties (Figure 4). As is evident in Figure 4, the stress concentration would be over the entire model when there is a fixed interface as the allocated joint properties between the coal and overburden. This is because of the fact that there is no slip between the engaged surfaces. On the other hand, a local stress concentration was observed in both models where CS and CY joints were specified with slip joint properties between the major surfaces. Individually, the stress concentration is located at the edges of the model in which the slip direction was entirely restrained in that direction due to the possible particular geological structure of the mine.

A comparison between the strain energy as well as the kinetic energy due to the different joint properties (i.e., the fixed joint properties, the CS joint properties and the CY joint properties) is presented in Figures 5 to 7. Figure 5 presents the relationship between the kinetic and strain energy when there is a fixed condition between the coal and overburden layers. As expected, it is evident from this figure that the strain energy is higher than (almost 4.5 times) the kinetic energy due to the lack of movement between the simulated layers, which indicates that the strain energy can be notably stored inside the strata layers rather than releasing as a kinetic energy due to the lack of the slip between different layers.
Both Figures 6 and 7, where the shear stresses between the joints are a function of slip between the layers, demonstrate that the kinetic energy is significantly higher (over 9.45 times) than the strain energy due to the movement between different layers. This finding is important as it verifies the mechanism of how the stored energy can be released into the different parts of the rock/coal interface.
From the above it is reasonable to conclude that in the burst-prone zones strata flexibility would be one of the critical considerations rather than only the strength and stiffness of the layers and joint properties. The kinetic energy, which can generally be transferred into the rock mass, can fully or partially be released from the strain energy which is stored in the rock mass. Thus, the source of the discussed strain energy may be significantly dependent on the geological structures (e.g. joint mechanical properties). Usually, the rock mass surrounding coal seams consists of considerable discrete layers. Therefore, it is possible that a significant amount of this strain energy can either be converted to active kinetic or passive thermal energy in different layers and it can lead to generating a large displacement as well as degradation of the rock mass which in turn might be highly distractive. Provided that

\[
\frac{E_{\text{Strain Energy}}}{E_{\text{Kinetic Energy}}} = m \quad \text{if} \quad m \geq 1
\]

then there is a tie or fixed joint between the layers. On the other hand, when \( m \leq 1 \) then there is a flexible joint between the simulated layers. This simple assessment can help determine rockburst-prone zones. This finding confirms that the numerical modelling as a robust tool can provide a reliable procedure to determine high-risk zones where a severe coal burst might occur. It is of note that the energy based design approach is a novel procedure when evaluating performance of mining structures. This approach can also be significantly extended by involving further key parameters such as energy dissipations due to the material damping between rock-mass layers as well as computing induced internal and external work as a results of relative

**Figure 6:** Strain and kinetic energies using CS joint properties at different computing times

**Figure 7:** Strain and kinetic energies using CY joint properties at different computing times
movements between the layers. It is however appreciated that numerical modelling may not be the solution for every mine due to the complexity involved.

CONCLUSIONS

An assessment of strain energy and kinetic energy before and during excavation can help to assess the likelihood of a violent failure. In this paper, bord-and-pillar mining layouts were modelled based on the different joint properties. It was concluded that continuously yielding joint properties presented by CY result in more energy release and thus have a significant influence on the of failure mode. Therefore, the rock mass failure mode with different joint properties might be critically affected by the transmission of energy between the layers. Furthermore, full scale simulations are suggested to gain a better understanding of the interaction between the key elements that govern the failure mode, as well as the energy momentum that builds up between the major layers.

ACKNOWLEDGEMENT

This work was supported by an award under the Merit Allocation Scheme on the NCI National Facility at the ANU and part of the computational services used in this work were provided by Intersect Australia Ltd.

REFERENCES


GEOTECHNICAL RISK ASSESSMENT AND SELECTION OF SUITABLE ROADHEADERS FOR TABAS COAL MINE PROJECT, IRAN

Reza Mohammadzadeh\(^1\) and Arash Ebrahimabadi\(^2\)

**ABSTRACT:** Geotechnical risks play an important role in the selection of tunneling method and machine types. In order to prevent some geological and geotechnical hazards during tunneling, selecting the most appropriate tunneling machine is very important. In this research study the main geotechnical hazards of tunneling by roadheader with their common mitigation measures are discussed. At present, three roadheader are excavating the three drifts of the Tabas coal mine in Iran. Hence, these drifts were used in this research study. The risk assessment matrix method in these tunnels showed that there is a lower risk of tunneling by roadheader than utilising the drilling and blasting method. The determined geotechnical risk assessment showed that the transverse cutter head was more suitable in comparison to a longitudinal cutter head for such tunnels. Then by using an instantaneous cutting rate model, the cutter head power of the roadheader was determined. According to the model, a lightweight roadheader with a cutter head power of about 80-110 KW and 40 t weight are suitable. Finally, four types of manufactured roadheader with similar specifications were selected.

**INTRODUCTION**

Nowadays, among mechanical miners, roadheader are extensively used in mining and civil industries. In the 1960s, use of roadheader for mechanized tunneling began and by the end the 1970s, it gained considerable acceptance worldwide. In the past decade, the development of roadheader in tunneling has reached a cutting level not only in soft rock but also in medium to hard rock.

The increasing use of such machines in tunneling also demonstrates some problems and limitations in hard and abrasive rock as well as, in some cases, in soft rock formations. (Thuro *et al*., 1998) presented main geological and geotechnical parameters affecting performance of roadheader. Moreover, there are some research works focusing on performance prediction of roadheader. (Bilgin *et al*., 2004), (Ebrahimabadi *et al*., 2011a, 2011b, 2012, 2015) developed several models to predict the performance of roadheader through different approaches. AITES-ITA working Group (2009) presented some standards for selection of the most appropriate tunneling techniques and machines for excavation of tunnels and underground spaces. (Acaroglu *et al*., 2006) introduced a model for selection of roadheader by AHP approach, but the geological and geotechnical hazards in the process of selection have not been considered. Only few studies focusing on risk assessment and management in tunnels and underground spaces have been reported (Shahriar *et al*., 2008). This is undoubtedly due to the complexity and frequency of effective decision variables. Hence, in this study, the main geotechnical hazards of tunneling using roadheader and their common mitigation measures in Tabas coal mine in Iran are identified and a model for selecting the most appropriate roadheader is developed. This involves: collecting available geological and geotechnical data of the case study, Investigation and analysis of geotechnical hazards in Tabas coal mine tunnels. Selection the most appropriate tunneling method using a risk assessment index, geotechnical risk assessment for proper selection of the roadheader cutter head. Optimum selection of roadheader should be based on a practical model to predict the Instantaneous Cutting Rate (ICR) of the roadheader (Bilgin model) and comparing the parameters of selected roadheader with common manufactured roadheader.

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\(^2\) Department of Mining Engineering, Qaemshahr Branch, Islamic Azad University, Qaemshahr, Iran.
MAJOR PARAMETERS AFFECTING MACHINE SELECTION

When carrying out a multiple criteria decision-making process such as selecting a proper roadheader, the criteria that will affect the selection process should be determined beforehand. Roadheader manufactured with specified weight, power and size so that some parts of them cannot be modified or changed easily. This is why special attention should be paid to consider main selection criteria. In this stage, there is no need to involve those criteria which can be further modified. For more clarification, since the users can change cutterheads or picks on the cutter heads easily, the cutter head design parameters may not be taken as a criterion. The roadheader are classified based on machine weight, cutter head power light to heavy duty machines, which are capable of covering face area up to 45 $\text{m}^2$ and they are able to excavate rock formations with compressive strength of 20-140 MPa. Machines’ specifications are listed in Table 1. Figure 1 is also a useful guide to select a roadheader based on rock strength, tunnel cross section, required weight and power of the machine.

<table>
<thead>
<tr>
<th>Roadheader Class</th>
<th>Weight (t)</th>
<th>Cutter head Power (kW)</th>
<th>RH with standard cutting rate</th>
<th>RH with extended cutting rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>8-40</td>
<td>50-170</td>
<td>~25</td>
<td>~40</td>
</tr>
<tr>
<td>Medium</td>
<td>40-70</td>
<td>160-230</td>
<td>~30</td>
<td>~60</td>
</tr>
<tr>
<td>Heavy</td>
<td>70-110</td>
<td>250-300</td>
<td>~40</td>
<td>~70</td>
</tr>
<tr>
<td>Extra Heavy</td>
<td>&gt;100</td>
<td>350-400</td>
<td>~45</td>
<td>~80</td>
</tr>
</tbody>
</table>

Table 1: Classification of Roadheader (Ratan Raj Tatiya 2013)

![Figure 1: Indicative diagram for Roadheader selection in accordance with machine weight, power, rock strength and operating condition (Ratan Raj Tatiya 2013).](image)

GEOTECHNICAL HAZARDS IN TUNNELING

A geotechnical hazard in a mechanized tunneling can be defined as a difficult ground condition where the selected machine cannot operate as well as expected performance. Geotechnical hazards are often appeared because of insufficient geological–geotechnical investigations, as well as leading to serious and even catastrophic consequences during tunneling operation. Even though a certain degree of
geotechnical hazard is expectable using any kind of tunneling machines, selecting the appropriate one will result in minimum hazards. Main geotechnical hazards of tunneling with roadheader and common mitigation measures are illustrated in Table 2.

<table>
<thead>
<tr>
<th>Geotechnical Hazards</th>
<th>Mitigation measures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cuttability (Hardness, abrasivity)</td>
<td>• Use of roadheader with replaceable cutter head</td>
</tr>
<tr>
<td></td>
<td>• Use of resistant tungsten carbide bits</td>
</tr>
<tr>
<td></td>
<td>• Use of water jet assistance during cutting</td>
</tr>
<tr>
<td></td>
<td>• Applying higher torque</td>
</tr>
<tr>
<td>Roof, walls and face instability</td>
<td>• Use of support systems such as rock bolt, steel arches, shotcrete</td>
</tr>
<tr>
<td></td>
<td>• Pretreatment via grouting</td>
</tr>
<tr>
<td></td>
<td>• Tunnel lining with precast concrete segments</td>
</tr>
<tr>
<td></td>
<td>• Use of shielded roadheaders</td>
</tr>
<tr>
<td>Mixed ground condition</td>
<td>• Selective excavation in tunnel face</td>
</tr>
<tr>
<td></td>
<td>• Change cutter head</td>
</tr>
<tr>
<td>Water inflow</td>
<td>• Pre-injection</td>
</tr>
<tr>
<td></td>
<td>• Drainage</td>
</tr>
<tr>
<td></td>
<td>• Freezing</td>
</tr>
<tr>
<td></td>
<td>• Probe drilling</td>
</tr>
<tr>
<td>Clay- soft ground</td>
<td>• Ground improvement</td>
</tr>
<tr>
<td></td>
<td>• Drainage</td>
</tr>
<tr>
<td></td>
<td>• Flooring</td>
</tr>
<tr>
<td>Mucking</td>
<td>• Use of proper haulage system</td>
</tr>
<tr>
<td></td>
<td>• Use of a longitudinal cutter head</td>
</tr>
</tbody>
</table>

If occurrence of these hazards were foreseen before the beginning of the excavation operation, these mitigation measures, as preventive methods, would minimize or eliminate the geotechnical risks. Some of recent unsuccessful mechanized tunneling projects using roadheader along with their geological conditions and geotechnical hazards during construction are listed in Table 3 and the proper excavation techniques are shown in Table 3.

**DESCRIPTION OF TABAS COAL MINE PROJECT**

Tabas coal mine, the largest and unique fully mechanized coal mine in Iran, is located in central part of Iran near the city of Tabas in Yazd province and situated 75 km far from southern Tabas. The mine area is a part of Tabas-Kerman coal field. The coal field is divided into 3 parts in which Parvadeh region with the extent of 1200 Km² and 1.1 billion tons of estimated coal reserve is the biggest and main part to continue excavation and fulfillment for future years. The coal seam has eastern-western expansion with reducing trend in thickness toward east. Its thickness ranges from 0.5 to 2.2 m but in the majority of conditions it has a consistent 1.8 m thickness. Room and pillar and also long wall mining methods are considered as the main excavation methods in the mine. The use of roadheaders in Tabas coal mine project was a consequence of mechanization of the work. Coal mining by the long-wall method with powered roof supports makes rapid advance of the access roads necessary. On the other hand, the two alternatives for mining very thick coal seams, i.e. room-and-pillar and long wall in flat seams, also make the use of roadheader driving galleries in the coal seams necessary (Ebrahimabadi et al., 2011).

**GEOTECHNICAL HAZARDS IN TABAS COAL MINE TUNNELS**

**Cuttability in hard and abrasive rock formations**

The most important parameters that should be considered are to investigate whether machine is economically capable of excavating the hard and abrasive faces?
Highly abrasive rocks could contribute to fast bit consumption as well as cutter head replacement which make the machine advance problematic from both a time and economical points of view. Geomechanical parameters of rock formations in the tunnels route are presented in Table 4. As can be seen from Table 4, all rocks expect sandstone have a compressive strength less than 50 MPa.

According to international tunneling and underground space association (ITA-AITES), the definition of hard rock and soft ground are predicated to the rocks with 50-100 and 5-50 MPa, respectively. In the majority of cases, tunnels faces consist of soft ground conditions. Therefore, the possibility of encountering difficult conditions is very low.

Table 3: Summery of some tunnels with inappropriate excavation techniques

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>L (km)</th>
<th>D (m²)</th>
<th>Geological conditions</th>
<th>Hazards</th>
<th>Selected roadheader</th>
<th>Proper technique</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Altenberg (Germany)</td>
<td>320</td>
<td>92</td>
<td>Dense quartzites, argillaceous slates and volcanic rock in different stages of weathering</td>
<td>Enormous bit consumption and exceptional poor penetration</td>
<td>Dosco MK2A 300 kW</td>
<td>Drilling and blasting</td>
<td>Thuro (1998)</td>
</tr>
<tr>
<td>Meisten (Germany)</td>
<td>1684</td>
<td>140</td>
<td>fanglomerates, sandstone, clay-siltstones, layers of calcite</td>
<td>High compressive strength in calcite layers, fanglomerates, clay-siltstone, very widely spaced joints and bedding plane</td>
<td>Paurat E242B 300kW</td>
<td>Drilling and blasting</td>
<td>Thuro &amp; Plinninger (1998)</td>
</tr>
<tr>
<td>Zeulendorf (Germany)</td>
<td>2400</td>
<td>11</td>
<td>Quartzites of high compressive strength, faulted and fractured material, fault gauge and heavy jointing</td>
<td>High bit consumption, abrasive hard rock</td>
<td>Atlas Copco Eickhoff ET 120 132kW</td>
<td>Change from longitudinal cutter head to Transverse cutter</td>
<td>Deketh (1996)</td>
</tr>
<tr>
<td>Nuremberg (Germany)</td>
<td>3300</td>
<td>-</td>
<td>Sandstone, clay-siltstone, very hard dolomitic, calcitic calcare</td>
<td>Problem in mucking system related to the amount of clay and silt, highly water permeable</td>
<td>Atlas Copco Eickhoff ET 380 200kW</td>
<td>Drilling and blasting</td>
<td>Thuro &amp; Plinninger (1998)</td>
</tr>
<tr>
<td>Emamzadeh Hashem (Iran)</td>
<td>3180</td>
<td>79-93</td>
<td>Hard quartzites, sandstone quartzites, soft coal layers, weak shale layer</td>
<td>Diversity layers in face, machine subsidence in shale-coal layers, water permeable</td>
<td>Paurat T2-11 Transverse cutter head 422 kW</td>
<td>Drilling and blasting</td>
<td>Forough (2003)</td>
</tr>
<tr>
<td>Gavoshan (Iran)</td>
<td>20000</td>
<td>5.5-7.5</td>
<td>Unweathered felses and calcretemann, sand shale and thin sandstone</td>
<td>High percentage of silica and abrasive rocks, mucking problem, machine subsidence</td>
<td>Paurat T2-11 Transverse cutter head 422 kW</td>
<td>Drilling and blasting</td>
<td>Forough (2003)</td>
</tr>
</tbody>
</table>

**Tunnels roof, walls and face instability**

In Tabas coal mine, tunnel faces include coal and coal measure rocks such as shale, siltstone and sandy siltstone with heavy jointing. On the other hand, structural instabilities could stem from the jointed rock masses.
According to Rock Mass Rating (RMR) analyses, the rock mass is ranked as poor class. The average stand up time for such tunnels is nearly 80 minutes. Thus, rock bolts with shotcrete in the roof and steel frame can be suggested as the initial support system.

Table 4: Summary of the geomechanical parameters of rock mass in Tabas coal mine tunnels

<table>
<thead>
<tr>
<th>Rock type</th>
<th>RQD (%)</th>
<th>Tensile strength (MPa)</th>
<th>Uniaxial Compressive strength (MPa)</th>
<th>Density (t/m$^3$)</th>
<th>RMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy siltstone</td>
<td>25-30</td>
<td>6.6</td>
<td>80-90</td>
<td>2.7</td>
<td>48</td>
</tr>
<tr>
<td>Siltstone</td>
<td>20-25</td>
<td>5.2</td>
<td>35-45</td>
<td>2.72</td>
<td>43</td>
</tr>
<tr>
<td>Shale</td>
<td>10≤</td>
<td>0.14</td>
<td>10-20</td>
<td>2.65</td>
<td>38</td>
</tr>
<tr>
<td>Coal</td>
<td>10≤</td>
<td>0.1</td>
<td>10-15</td>
<td>1.5</td>
<td>32</td>
</tr>
</tbody>
</table>

**Mixed ground condition**

There are various layers appearing in the tunnel cross-section due to the tunnel inclination the coal seam is combined with other formations, which makes soft and hard layers be located beside rock bed in some sections. For instance the presence of coal and shale layers in the vicinity of sandy siltstone would make mixed condition in the tunnel face. Figure 2 illustrates several views of mixed ground condition in the drift galleries.

**Figure 2: General views of mixed ground condition in tunnels route**

**Water inflow**

The presence of groundwater may worsen the situations. The presence of water in crushed zones in the vicinity of fault zones may cause failures.

For example, in apparently good rock formations (such as soft siltstones and shale) with efficient cutting performance, even a low water inflow can lead to a total disaster. Under these circumstances, the cutter head may be smudged by clay.

**Clay – soft ground**

With regard to soft layers of tunnel and other zones mentioned in Table 5, the likelihood of shale and siltstone located in tunnel floor are expected. Using a roadheader in such soft ground makes difficulties. In some zones, two or more layers located in the floor which cause instability of the roadheader. The machine may sink and have movement problems in such soft ground. In some sections, weak coal and shale layers in the floor decrease the machine performance and rate of cutting. Also the clay layers in
the presence of water caused additional deterioration of the requiring ground improvement. Therefore tunnel alignment should be positioned somehow that the hard rocks remain at the floor at the first stage of tunnel design.

**Mucking**

According to Table 5, the inclinations of tunnels are moderate. There are some problems in haulage and loading of cuttings in the tunnel face due to G-force, because at the bottom of the tunnel face, dumped material cannot be removed easily. In some cases, soft clay siltstone forms mud with a distinct amount of water and cannot be removed by the roadheader’s haulage system.

With respect to cutter head type, the transverse cutterhead has better performance in mucking where it can improve the transfer the rock fragments on to the apron by 80% (Ratan Raj Tatiya 2013).

<table>
<thead>
<tr>
<th>Zone</th>
<th>Inclination (Degree)</th>
<th>Tunnel floor rocks</th>
<th>Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tunnel 1</td>
</tr>
<tr>
<td>1</td>
<td>7-8</td>
<td>Siltstone and sandy siltstone</td>
<td>115-120</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Coal</td>
<td>211-223</td>
</tr>
<tr>
<td>3</td>
<td>11.3 – 12.9</td>
<td>Siltstone and sandy siltstone</td>
<td>211-280</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>Mixed</td>
<td>270-320</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>Coal</td>
<td>223-270</td>
</tr>
</tbody>
</table>

**SELECTION OF EXCAVATION METHOD**

With regard to Tabas mine tunnels which are mostly located inside the soft coal layers having an uniaxial compressive strength of nearly 10-20 MPa, two methods are candidates for excavation: conventional drilling and blasting method or mechanical excavation using roadheader.

Accordingly, the most suitable excavation method using geotechnical risk assessment will be selected.

**GEOTECHNICAL RISK ASSESSMENT IN TUNNELING PROJECT**

Geotechnical risks in tunnelling generally deal with potential hazardous geotechnical conditions that could unfavourably affect a tunnelling project and might – in the worst case – cause human fatalities. Additional consequences of less significance could include damage to equipment, interruption of work, contractual claims, all of which eventually lead to delays of the project schedule and/or increase of the project costs.

Risk assessments as well as the effects of risk mitigation measures are two essential elements during each early engineering phase. The level of risk for each hazard can be determined by finding its likelihood of occurrence and considering its consequence, then multiplying them as the following formula shows:

\[
\text{Risk} = \text{Likelihood} \times \text{Consequence}
\]

(1)

As a general rule, the likelihood of occurrence and consequence can be divided into arbitrary levels. Here, in order to get more precise results, the five-level of each one was used. The rating of likelihood and consequence is presented in Tables 6 and 7. Combining the likelihood rating and the consequence rating results in a risk index of between 1 and 25 for any given risk, presented in Table 8.

**Geotechnical risk assessment for drilling and blasting method**

Table 9 shows the most important geotechnical hazards and risks using the drilling and blasting method associated with geological and geotechnical data in this field. According to Table 9, the risk level of the
drilling and blasting method is at the border of tolerability while the mitigation measures or removing risk is highly required.

Table 6: Rating of likelihood of hazards occurrence

<table>
<thead>
<tr>
<th>Likelihood</th>
<th>Rating</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Improbable</td>
<td>1</td>
<td>Event is extremely unlikely to occur once</td>
</tr>
<tr>
<td>Remote</td>
<td>2</td>
<td>Event is unlikely to occur once</td>
</tr>
<tr>
<td>Probable</td>
<td>3</td>
<td>Event is likely to occur at least once</td>
</tr>
<tr>
<td>Expected</td>
<td>4</td>
<td>Event is likely to occur more than once but infrequently</td>
</tr>
<tr>
<td>Frequent</td>
<td>5</td>
<td>Event is likely to occur frequently</td>
</tr>
</tbody>
</table>

Table 7: Rating of consequence of hazards occurrence

<table>
<thead>
<tr>
<th>Consequence</th>
<th>Rating</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>1</td>
<td>Event does not cause delay or damage</td>
</tr>
<tr>
<td>Moderate</td>
<td>2</td>
<td>Event causes minor damage and/or delay up to 2 days</td>
</tr>
<tr>
<td>Serious</td>
<td>3</td>
<td>Event causes repairable damage and/or delays up to 1 week</td>
</tr>
<tr>
<td>Critical</td>
<td>4</td>
<td>Event causes significant repairable damage and/or delays between 1 and 2 weeks</td>
</tr>
<tr>
<td>Catastrophic</td>
<td>5</td>
<td>Event causes irreparable damage and/or delays greater than 2 weeks</td>
</tr>
</tbody>
</table>

Table 8: Risk index for any given risk

<table>
<thead>
<tr>
<th>Risk</th>
<th>Index</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>1–4</td>
<td>Risk is tolerable without any mitigation</td>
</tr>
<tr>
<td>Medium</td>
<td>5–9</td>
<td>Risk is moderately tolerable. Mitigation may be needed</td>
</tr>
<tr>
<td>High</td>
<td>10–15</td>
<td>Risk is at the border of tolerability. Mitigation should be identified and implemented to reduce risk</td>
</tr>
<tr>
<td>Very high</td>
<td>16-25</td>
<td>Risk is intolerable. Mitigation that reduces risk must be implemented</td>
</tr>
</tbody>
</table>

Table 9: Geotechnical risk indices for using drilling and blasting method

<table>
<thead>
<tr>
<th>Geotechnical Hazards</th>
<th>Risk</th>
<th>Occurrence likelihood</th>
<th>Consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jointed rock mass</td>
<td>Soft ground, Roof, walls and face instability</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Fault zones</td>
<td>Soft or mixed ground condition</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Water inflow</td>
<td>Water permeability in rock pores during drilling</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Rock mass porosity</td>
<td>Roof, walls and face instability</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Tunnel ventilation</td>
<td>Explosion risks</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Using explosive materials</td>
<td>Unwanted explosion</td>
<td>2</td>
<td>5</td>
</tr>
</tbody>
</table>

Geotechnical risk assessment in tunneling using roadheader

Risk level of geotechnical hazards in mining and tunneling by roadheader are shown in Table 10. In order to select a proper roadheader with an appropriate cutterhead, geotechnical hazards considering risk level, likelihood and consequences for both transverse and longitudinal cutterheads are evaluated and compared in Table 10.

By comparing risk level between the two methods and employing appropriate methodologies to mitigate hazards and decrease risk level, it would be clear that using roadheader is more suitable and safer than the drilling and blasting method. Since there is a small difference between two kinds of cutterheads in relation with risk level, using transverse cutterhead seems to be better.
In order to select an appropriate and efficient roadheader, machine parameters based on the main design parameters such as machine weight classification and cutterhead power should be considered, as well as geometric and geotechnical parameters of the task.

In accordance with the geological and geotechnical conditions, the desired selected machine should have conditions such as light weight with the minimum applied force on the ground, high rate of penetration, operating under difficult ground condition and capability of changing the cutter head, easily.

### Table 10: Geotechnical risk indices in case of using roadheader

<table>
<thead>
<tr>
<th>Geotechnical Hazards</th>
<th>Occurrence likelihood</th>
<th>Consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Transverse cutter head</td>
</tr>
<tr>
<td>Cuttability</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>(Hardness, abrasivity)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof, walls and face instability</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Mixed ground condition</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Water inflow</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Clay- soft ground</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Mucking</td>
<td>4</td>
<td>2</td>
</tr>
</tbody>
</table>

### OPTIMUM SELECTION OF A ROADHEADER FOR EXCAVATION

**Optimum selection of a cutterhead**

There are several methods to calculate the optimum cutterhead power. The best way is to conduct a series of full-scale linear cutting tests. Another method is an empirical method proposed by Bilgin which is applied in this research. This model is used to predict the net rate of cutting.

The cutter head power is determined by equations (2) and (3), this model was tested and developed by Bilgin based on using different kinds of cutterheads (Transverse and longitudinal) and mixed rock masses and is known as an efficient method for faulty and weak zones according to many of mining and tunnelling projects case studies, the equations are as follows:

\[
I_{CR} = 0.28 \times H_P \times (0.974)^{R_{MCI}}
\]  
\[R_{MCI} = U_C_S \times \left(\frac{R_Q D}{100}\right)\]  

Where:

- \(I_{CR}\) = instantaneous cutting rate (m³/h)
- \(U_C_S\) = uniaxial compressive strength (MPa)
- \(R_{MCI}\) = rock mass cutting rate
- \(H_P\) = cutterhead power (Kw)
- \(R_Q D\) = rock quality designation (%)

\(H_P\) can be calculated as bellow by equation (2)

\[H_P = \frac{I_{CR}}{0.28 \times (0.974)^{R_{MCI}}}\]  

It seems a 20 m³/h cutting rate in normal conditions in a tunnel includes coal layers can be considered as an acceptable cutting rate for instantaneous cutting. Rock Mass Cutting Rate (RMCI) and cutterhead power for various rock layers are shown in Table 11 by using equation 3, 4 based on various layers compressive strength and Rock Quality Designation (RQD), supposing that the mine has layers with
uniform structures and the face consists of coal, siltstone, sandy siltstone and shale layers with uniform structures.

Using equation 5, power in Kw can be calculated as below:

$$112\text{HP} \times 0.7457 = 83.5\text{ Kw}$$

(5)

Some other main parameters of roadheader can be obtained by using roadheader classification table (Table 1) and roadheader selection indicative diagram (Figure 1). Table 12 shows the main specifications of the suggested roadheader.

**Table 11: The cutterhead power based on uniaxial compressive strength of rocks**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>UCS (MPa)</th>
<th>RQD</th>
<th>RMCI</th>
<th>Power (HP)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>10</td>
<td>10</td>
<td>2.15</td>
<td>75.75</td>
</tr>
<tr>
<td>Coal</td>
<td>15</td>
<td>10</td>
<td>3.23</td>
<td>77.82</td>
</tr>
<tr>
<td>Shale</td>
<td>10</td>
<td>10</td>
<td>2.15</td>
<td>75.75</td>
</tr>
<tr>
<td>Siltstone</td>
<td>35</td>
<td>20</td>
<td>11.96</td>
<td>98.03</td>
</tr>
<tr>
<td>Siltstone</td>
<td>45</td>
<td>25</td>
<td>17.85</td>
<td>114.94</td>
</tr>
<tr>
<td>Sandy siltstone</td>
<td>80</td>
<td>25</td>
<td>31.74</td>
<td>165.28</td>
</tr>
<tr>
<td>Sandy siltstone</td>
<td>90</td>
<td>30</td>
<td>40.33</td>
<td>208.30</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td>112</td>
</tr>
</tbody>
</table>

**Table 12: The main parameters of suggested roadheader**

<table>
<thead>
<tr>
<th>Roadheader type</th>
<th>Light</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (ton)</td>
<td>8-140</td>
</tr>
<tr>
<td>Cutterhead power (Kw)</td>
<td>80-110</td>
</tr>
<tr>
<td>Max section (m²)</td>
<td>25</td>
</tr>
<tr>
<td>Max UCS (MPa)</td>
<td>60-80</td>
</tr>
</tbody>
</table>

**Roadheader selection among common models**

Finally, four models among manufactured roadheader that were extensively used in mining and tunnelling projects in the world, with specifications similar to the determined one and compatible with the mine conditions, were selected.

The selected machines with their specifications are shown in Table 13. Other parameters that can affect the machine performance and lead to selecting the appropriate machine are also presented in Table 13.

**Table 13: Selected roadheader and their specifications**

<table>
<thead>
<tr>
<th>Model</th>
<th>Dosco Overseas SL120</th>
<th>AtlasCopco-Eickhoff ET-110</th>
<th>Dosco Overseas MD1100</th>
<th>Voest-Alpine AM50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (ton)</td>
<td>33</td>
<td>25</td>
<td>31.5</td>
<td>24</td>
</tr>
<tr>
<td>Cutterhead power (Kw)</td>
<td>82</td>
<td>110</td>
<td>82</td>
<td>110</td>
</tr>
<tr>
<td>Total power (Kw)</td>
<td>165</td>
<td>185</td>
<td>157</td>
<td>170</td>
</tr>
<tr>
<td>Max cutting height (m)</td>
<td>4.1</td>
<td>4</td>
<td>4.2</td>
<td>2.4.8</td>
</tr>
<tr>
<td>Max cutting width (m)</td>
<td>2.4.3</td>
<td>5.3</td>
<td>2.7-5.7</td>
<td>4.8</td>
</tr>
<tr>
<td>Ground contact pressure (bar)</td>
<td>1.5</td>
<td>1.4</td>
<td>1.4-1.7</td>
<td>1.3</td>
</tr>
<tr>
<td>Speed (m/min)</td>
<td>13.8</td>
<td>0.5</td>
<td>7.2</td>
<td>6</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

Geotechnical hazards are considered as the most critical risks in mining and tunnelling industries, which always threaten the safety of miners and equipment. In order to mitigate and control them, a powerful
and dynamic management system for assessment of risks is needed. This system should be able to recognize and assess the risks and suggest the most relevant alternatives. The most important geotechnical risks in mining and tunnelling using roadheader are cuttability, roof, wall, face instability, mixed ground condition, water inflow, clay-soft ground and mucking. According to this research, the risk level between two methods of conventional drilling and blasting and tunnelling by roadheader was considered and application of roadheader is better and has a lower risk. With regard to risk levels of studied tunnels, using transverse cutterhead is more suitable than longitudinal type. According to researches on geometric and geomechanical parameters of encountered faces in tunnels of Tabas coal mine, DOSCO MD1100 roadheader was found to be the optimum selection.

Case studies show there are always some difficulties in tunnelling using roadheaders in the Tabas coal mine project which have caused delay in operation, low performance of machine and sometimes using changing in to the method to drilling and blasting. To eliminate such problems, some suggestions are made: If possible, the tunnel alignment should be designed somehow that the hard rocks remain at the floor. Apply temporary ground improvement on soft clayey ground which has a positive effect on roadheader’s performance. Drainage, freezing, pre injection can help to decrease water inflow into the tunnel. Availability of two kinds of cutterhead (transvers and axial) in order to change them on time in the case of encountering different geological conditions in a tunnel also enhances roadheader performance in coal measure rocks.

Collecting the more exploratory and geological data of a tunnel by experts helps to select the optimum and desired machine with proper equipment for a continuous excavation process and fewer difficulties.

REFERENCES


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CAUSES OF OVERBREAK IN TUNNELING: A CASE STUDY OF THE ALBORZ TUNNEL

Mohammad Farouq Hossaini¹, Mohammad Mohammadi², Jebreil Ghadimi² and Alireza Abbasi³

ABSTRACT: Drilling and blasting is widely used in underground excavation projects, where the amount of damage to the surrounding rock mass is crucially important due to its impact on the safety of working environment and operational costs. The causes of overbreak are categorized into three groups namely: geological parameters, drilling accuracy and charging parameters. The present paper focuses on the special case of the Alborz Tunnel of Iran where a discontinuity surface located above the tunnel contour line caused excessive amounts of overbreak in the study area. After introducing the disconformity surface above the tunnel contour line, its impact on the occurrence of excessive amounts of overbreak is discussed. Possible case scenarios for future excavations are pointed out and the problems which may be encountered in each case scenario are predicted. Also, the impact of this special situation on the difficulties faced in working with rock mass classification systems is discussed.

INTRODUCTION

As a result of damage to surrounding rock mass, overbreak can be quantified as the extra cost of additional removal of muckpile and installation of extra support tunnel face with an area of seventy square metres is located in sandstone. After a sixty-metre length of advance in sandstone, a layer of weak argillite appeared above the tunnel axis having a weak bond with sandstone which led to occurrence of excessive overbreak. Field examination of the sandstone and argillite layers revealed the existence of a disconformity surface just above the tunnel axis which is shown in Figure 1. The estimated argillite thickness above the tunnel axis is between two and three metres based on the drilled holes for rock bolting and penetration speed of the bit. The special case of this cause of overbreak is going to be discussed in this paper.

Figure 1: Location of the Alborz Tunnel

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³Laniz Consulting engineers, the Alborz Tunnel project, Alborz province, Iran,
OVERBREAK OCCURRENCE

The main reason for the occurrence of excessive overbreak in the Northern face of the Alborz Tunnel in the study area is due to the existence of a disconformity surface just above the tunnel contour as depicted in Figure 2. The current excavation face is 635 metres into the tunnel from the Northern portal. However, the occurrence of overbreak which led to discovery of the disconformity surface started from 575 metres into the tunnel from the Northern Portal as shown in Figure 3.

Due to the dip and dip direction of the disconformity surface, the occurrence of overbreak in the start point is almost 1.5 metres above the contour line of the tunnel (Figure 4) and decreases gradually till the point where the disconformity surface reaches the contour line of the tunnel.

Figure 2: Argillite and sandstone layers along with disconformity surface
POSSIBLE SCENARIOS OF CAUSING OVERBREAK AS EXCAVATION ADVANCES

As discussed, the special geological condition namely existence of a disconformity surface above the contour line of the tunnel was the main reason for occurrence of overbreak in the Northern excavation face of the Alborz Tunnel.

The distance of the surface of the disconformity from the contour line (overbreak area) is gradually decreasing with respect to the dip of the disconformity surface. Therefore, the first scenario can be the gradual decrease of overbreak area as the excavation face advances. However, there will be overbreak occurrences till the surface of disconformity reaches the contour line. Then, the main problem will be the existence of a weak layer of argillite in the tunnel crown. Finally, the argillite layer will pass through the contour line to gradually climb down the excavation face till it disappears. The occurrence of overbreak in this possibility is shown in Figure 5. The second scenario is that the argillite layer could be the bottom of a fold which means that the argillite layer is going to gradually increase its distance from the contour line causing a gradual increase of overbreak as the excavation advances. In this case, the exact values of overbreak may occur in reverse order (gradual increase) from the axis of the fold till the argillite layer separates from the contour line by 1.5 metres or more. However, this possibility is very unlikely to happen as there are no joint sets in the argillite layer.

Figure 3: a) Existence of disconformity surface just above the tunnel contour b) Occurrence of excessive overbreak due to existence of disconformity surface above contour line

Figure 4: the start point of overbreak occurrence
Incompetency of rock classification systems in the rock formations consisted of alternation of weak and strong layers of rocks is discussed by Gonbadi et al. (2009). They reported the incompetency of the RMR system in the Shemshak formation and proposed a solution based on the modeling results taking the thickness and orientation of weak layers into account. In the study area of this paper, in the distance of 60 metres and more from the excavation face, the tunnel face was situated in the sandstone and before the occurrence of overbreak as shown in Figure 3. At this stage the existence of the weak argillite layer was unknown. Thus, the RMR rock classification was resorted to by the authors. According to RMR classification the surrounding sandstone was found to be classified as fair rock (class III).

Based on this classification system, for a ten-metre span tunnel, the advance in the top heading would be 1.5 to 3 metres. The support system will consist of systematic bolting of 4 metres long bolts, spaced 1.5 to 2 metres in the crown and walls. Wire mesh along with 50 to 100 mm of shotcrete in crown and 30 mm inside would secure the tunnel. No installation of steel sets would be needed (Bieniawski 1989). This combination of the support system was successful in keeping the working environment a safer place. However, the most important feature in this particular case was the excessive amount of overbreak rather than any long term stability problem. No solution to such a problem is offered by classification systems.

CONCLUSIONS

The rate of overbreak occurring in construction of underground structures is one of the most important parameters to be dealt with. The occurrence of overbreak leads to imposing additional costs to the project as well as decreasing the safety of the working environment. An exciting situation was encountered during the excavation of the Northern end of the Alborz Tunnel. The existence of a disconformity surface just above the tunnel contour leading to excessive amounts of overbreak is discussed in this paper and the followings are concluded:

- The existence of a disconformity surface just above the contour line is explored and demonstrated.
- The uncommon behavior of surrounding rock mass was explained by identification of the
disconformity surface.

- The role of the disconformity surface on the occurrence of overbreak and its amount is discussed and predictions are made for the occurrence of overbreak in the future excavations.
- Two possible case scenarios about the occurrence of overbreak are introduced and discussed separately.
- Problems with applying rock mass classification systems in such conditions are highlighted.

REFERENCES


DEFORMABILITY MODULUS OF JOINTED ROCKS, LIMITATION OF EMPIRICAL METHODS AND INTRODUCING A NEW ANALYTICAL APPROACH

Mahdi Zoorabadi¹ & ²

ABSTRACT: Deformability modulus of jointed rocks is a key parameter for stability analysis of underground structures by numerical modelling techniques. Intact rock strength, rock mass blockiness (shape and size of rock blocks), surface condition of discontinuities (shear strength of discontinuities) and confining stress level are the key parameters controlling deformability of jointed rocks. Considering cost and limitation of field measurements to determine deformability modulus, empirical equations which were mostly developed based on rock mass classifications are too common in practice. All well-known empirical formulations dismissed the impact of stress on deformability modulus. Therefore, these equations result in the same value for a rock at different stress fields. This paper discusses this issue in more detail and highlights shortcomings of existing formulations. Finally it presents an extension to analytical techniques to determine the deformability modulus of jointed rocks by a combination of the geometrical properties of discontinuities and elastic modulus of intact rock. In this extension, the effect of confining stress was incorporated in the formulation to improve its reliability.

INTRODUCTION

The deformability modulus of jointed rock mass is key parameters which is required for numerical and analytical analysis of structures in or on the rock mass. It is defined as the ratio of stress to corresponding strain (Figure 1) during loading of a rock mass, including elastic and inelastic behaviour (Ulusay and Hudson 2006).

Figure 1: Typical loading response of jointed rock

Estimation of deformability modulus is a significant challenge for rock engineers. Although field tests and measurement are better methods to determine this parameter, they are costly and imply notable operational difficulties. In the most practical applications, empirical equations which developed on the basis of case studies and rock mass classification systems are common tools to estimate deformability modulus. Over the years, many empirical equations were introduced by researchers and engineers by

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correlating field measurements with well-known classification systems such as RMR, Q, GSI, RMi, and so on. In Table 1, a list of these equations was presented.

Table 1: Empirical equations using RMR and GSI (Shen et al., 2012)

<table>
<thead>
<tr>
<th>Input parameters</th>
<th>Empirical equations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group 1 RMR</strong></td>
<td></td>
</tr>
<tr>
<td>Bieniawski (1978)</td>
<td>$E_r = 200RMR - 100; RMR &gt; 50$</td>
</tr>
<tr>
<td>Scarfino and Penczka (1983)</td>
<td>$E_r = 10^{(0.77RMR - 10)}$</td>
</tr>
<tr>
<td>Mohorut (1992)</td>
<td>$E_r = 0.1RMR^{1.1} $</td>
</tr>
<tr>
<td>Read et al. (1999)</td>
<td></td>
</tr>
<tr>
<td>Nicholson and Bieniawski (1990)</td>
<td>$E_r = 0.015(0.001RMR^2 + 0.006RMR + 0.001)$</td>
</tr>
<tr>
<td>Mitt et al. (1994)</td>
<td>$E_r = 4(0.51 - (0.14 + (RMR - 100)))$</td>
</tr>
<tr>
<td>Somozio et al. (2006)</td>
<td></td>
</tr>
<tr>
<td>Hoek et al. (2002)</td>
<td>$E_r = 10^{(0.77 - RMR) - (0.3RMR/100)}$</td>
</tr>
<tr>
<td>Hoek and Diederich (2006)</td>
<td>$E_r = 10^{(0.77 - RMR) - (0.3RMR/100)}$</td>
</tr>
<tr>
<td><strong>Group 2 RMR and $E_i$</strong></td>
<td></td>
</tr>
<tr>
<td>Hoek and Brown (1997)</td>
<td>$E_r = (1 - 0.50)(10000)^{E_i/2}$, $E_r &gt; 100$ MPA</td>
</tr>
<tr>
<td>Ei(MPA) = $10^{2(10000)^{E_i/2}}$</td>
<td></td>
</tr>
<tr>
<td>Zoorabadi (2010)</td>
<td></td>
</tr>
<tr>
<td>Saffar et al. (2004)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Hoek and Brown (2006)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td><strong>Group 3 GSI and $D$</strong></td>
<td></td>
</tr>
<tr>
<td>Hoek and Brown (1997)</td>
<td>$E_r = 0.02 + 2.17GSI + 0.001GSI^2$</td>
</tr>
<tr>
<td>Ei(MPA) = $10^{2(10000)^{E_i/2}}$</td>
<td></td>
</tr>
<tr>
<td><strong>Group 4 GSI, $D$ and $E_i$</strong></td>
<td></td>
</tr>
<tr>
<td>Carvalho (2004)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Saffar et al. (2004)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Hoek and Brown (2006)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td><strong>Group 5 GSI, $D$ and $\sigma_u$</strong></td>
<td></td>
</tr>
<tr>
<td>Hoek and Brown (1997)</td>
<td>$E_r = 0.02 + 2.17GSI + 0.001GSI^2$</td>
</tr>
<tr>
<td>Ei(MPA) = $10^{2(10000)^{E_i/2}}$</td>
<td></td>
</tr>
<tr>
<td>Zoorabadi (2010)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Saffar et al. (2004)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Hoek and Brown (2006)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td><strong>Group 6 GSI, $D$ and $\sigma_u$</strong></td>
<td></td>
</tr>
<tr>
<td>Hoek and Brown (1997)</td>
<td>$E_r = 0.02 + 2.17GSI + 0.001GSI^2$</td>
</tr>
<tr>
<td>Ei(MPA) = $10^{2(10000)^{E_i/2}}$</td>
<td></td>
</tr>
<tr>
<td>Zoorabadi (2010)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Saffar et al. (2004)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
<tr>
<td>Hoek and Brown (2006)</td>
<td>$E_r = 1.02 + 1.17RMR + 0.001RMR^2$</td>
</tr>
</tbody>
</table>

Deformability of jointed rocks is controlled by deformation of its intact rock blocks and discontinuities (Hoek and Brown 1997). Zoorabadi (2010) performed a parameter study on some of the existing empirical equations to explore the contribution of intact rock and rock mass condition to the deformability modulus estimated from those equations. It was found that in Hoek and Brown (1997) equation, intact rock properties (UCS) has a small contribution to the rock mass modulus. This condition was modified in Hoek and Diederichs (2006) equation (the most common equation in practice) which gives more contribution for intact rock property (Figure 2a, b).

![Figure 2a: Parameter study on Hoek and Brown (1997) and Hoek and Diederichs (2006) equations (Zoorabadi 2010)](image)

Stress dependency of deformability modulus which was not considered in empirical equation is the main shortcoming of all these equation and is the main objective of this paper. Deformability of rock discontinuities and rotation of rock block have a significant influence on deformability of jointed rocks located at ground surface where stress level in negligible. An applied normal stress on a rock fracture causes the fracture to close and decreases the aperture. The deformability of rock fractures due to normal stress has been studied intensively by several researchers. Goodman (1976) performed
laboratory tests and found a significant nonlinear relationship between applied stress and fracture closure. He also found that the nonlinear trend approaches an asymptote at high stress values (Figure 3).

Figure 3: Fracture closure due to applied normal stress (Goodman 1976)

Therefore, deformability of rock mass containing discontinuities would have different values at different depth or stress fields. In this paper an analytical method was applied to assess the variation of deformability modulus with depth and to investigate the contribution intact rock properties have on deformation modulus of rock masses.

ANALYTICAL FORMULATION TO ESTIMATE DEFORMABILITY MODULUS OF ROCK MASS

Analytical formulations use the elastic behaviour concept and combined mechanical properties and geometrical characteristics of rock discontinuities to determine the deformability modulus. Li (2001) used superposition principle and introduced an analytical approach to determine deformability modulus of a block of rock mass containing single or multiple joint sets. For multiple joint sets, his formulation for deformability modulus in loading direction has the following form:

\[
\frac{1}{E'} = \frac{1}{E} + \sum_{i=1}^{N} \frac{\cos^2 \theta_i}{S_i} \left( \frac{1}{k_{ni}} \cos^2 \theta_i + \frac{1}{k_{si}} \sin^2 \theta_i \right) \tag{1}
\]

where, \( E' \) is deformability modulus in loading direction, \( E \) is elastic modulus of intact rock, \( k_{ni} \) presents the normal stiffness of \( i \)th joint set, \( k_{si} \) is shear stiffness of \( i \)th joint set, \( S_i \) is spacing of \( i \)th joint set, \( N \) present the number of joint sets, and \( \theta_i \) is the angle between loading direction and normal vector of \( i \)th joint set (Figure 4).

Figure 4: A block of rock containing a single joint set loaded by uniaxial stress condition (modified from Ebadi et al., 2011)

As it was shown in Figure 3, normal stiffness represents the rate of change in normal stress with respect to discontinuity closure. Bandis et al., (1983) proposed the following empirical equation to estimate the normal stiffness of discontinuity under normal stress of \( \sigma_n \) as follows:

\[
k_n = k_{ni}(1 - \frac{\sigma_n}{V_m k_{ni} + \sigma_n})^{-2} \tag{2}
\]
where, $k_{ni}$ is initial normal stiffness, $\sigma_n$ is current level of applied normal stress, and $V_m$ is maximum closure of fracture. The following equation was introduced to calculate initial normal stiffness on the basis of Joint Roughness Coefficient (JRC), Joint Compression Strength (JCS) and initial aperture ($a_j$) of fracture as follows:

$$k_{ni} = -7.15 + 1.75JRC + 0.02\frac{JCS}{a_j} \quad (3)$$

The empirical equation to calculate the initial aperture of fracture has the following form:

$$a_j = \frac{JRC}{5} \left(0.2 \frac{\sigma_C}{JCS} - 0.1\right) \quad (4)$$

where, $\sigma_C$ represents the uniaxial compression strength of rock. Maximum closure of fracture is function of loading cycle and can be estimated from following empirical equation:

$$V_m = A + B(JRC) + C\left(\frac{JCS}{a_j}\right)^D \quad (5)$$

Constant values of $A$, $B$, $C$ and $D$ for each cycle of loading are listed in Table (2).

**Table 2: Constant values to estimate the maximum closure of rock fracture (Bandis et al., 1983)**

<table>
<thead>
<tr>
<th>Constant</th>
<th>1st Cycle</th>
<th>2nd Cycle</th>
<th>3rd Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>-0.2960 ± 0.1258</td>
<td>-0.1005 ± 0.0530</td>
<td>-0.1032 ± 0.0680</td>
</tr>
<tr>
<td>$B$</td>
<td>-0.0036 ± 0.0022</td>
<td>-0.0037 ± 0.0031</td>
<td>-0.0037 ± 0.0039</td>
</tr>
<tr>
<td>$C$</td>
<td>2.2410 ± 0.3504</td>
<td>1.0082 ± 0.2351</td>
<td>1.1350 ± 0.3261</td>
</tr>
<tr>
<td>$D$</td>
<td>-0.2450 ± 0.1086</td>
<td>-0.2301 ± 0.1171</td>
<td>-0.2510 ± 0.1029</td>
</tr>
</tbody>
</table>

Now, combination of Equations of 1 to 5 provide the capability to calculate the deformability modulus of jointed rocks when geometrical properties of discontinuities and field stress information are known. A real case located at Eastern of Australia (NSW) was used to investigate the impact of depth on the deformability modulus of jointed rock mass. In this case, the photogrammetry technique was implemented to determine the geometrical properties of discontinuities. In Figure 5 a photogrammetry results was presented.

The orientation and spacing of rock discontinuities for this case are listed in Table 3. To do a parameter study for this case, JRC and JCS of discontinuities were assumed as shown in Table 3. The elastic modulus of 16 GPa and UCS of 40 MPa were supposed for intact rock.
Table 3: Geometrical properties of rock discontinuities

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Dip</th>
<th>Dip/Dir</th>
<th>Spacing [m]</th>
<th>JRC</th>
<th>JCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>85</td>
<td>113</td>
<td>2.03</td>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>B</td>
<td>64</td>
<td>41</td>
<td>1.77</td>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>C</td>
<td>80</td>
<td>331</td>
<td>3.83</td>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>Bedding plane</td>
<td>24</td>
<td>156</td>
<td>4</td>
<td>10</td>
<td>30</td>
</tr>
</tbody>
</table>

Overcoring measurements and analysis of acoustic scanner data indicate a strong NW orientation for maximum horizontal stress. Furthermore, these tests show that the average ratio between maximum horizontal stress and minimum horizontal stress is \( \sigma_H / \sigma_h = 1.5 \). Since these tests had been performed at a limited number of depths, it was not possible to use them to estimate the magnitude of field stress variation with depth. Nemcik et al., (2005) presented statistical analysis of measured stress in Australia coal mines as Figure (6). The most probable trend in this dataset was used to determine the ratio between maximum horizontal stress and vertical stress at different depth. Magnitude and orientation of the maximum horizontal stress was considered as stress level that controls the normal and shear stiffness of discontinuities for this case.

The magnitude of the maximum horizontal stress was determined from Figure 6 and the angle between this stress component and discontinuities was used for calculations in Equation 1. In Figure 7, variation of calculated deformability modulus with depth is presented.

The deformability modulus of a block containing detected discontinuities at the ground surface (zero acting normal stress was assumed) was calculated to be 7.2 GPa. This value is around 0.45% of elastic modulus of intact rock and demonstrates the controlling impact of discontinuity deformation on deformability of a jointed rock. Deformability modulus of a this case increases significantly with depth increase. As it can be seen, just at 50 m depth, it would have a magnitude of 12.5 GPa which is 0.78% of the elastic modulus of intact rock. For depths deeper that 200 m, deformability modulus of a this rock mass would be more that 90% of the elastic modulus of intact rock. These results highlight a decreasing trend for discontinuity influence on the deformability of jointed rocks when depth increases.
Figure 7: Variation of deformability modulus with depth

With available data for spacing of discontinuities (S), the method introduced by Snomez and Ulusay (1999) can be used to determine the GSI of jointed rock for this case (Figure 8).

Figure 8: GSI table (Snomez and Ulusay 1999).
In this method the volume of rock blocks ($J_v$) between discontinuities is estimated by Equation 6. Then the result is used to calculate the Structure Rating (SR) value.

$$J_v = \sum_{j=1}^{n} \frac{1}{S_j}$$  \hspace{1cm} (6)

With joint surface condition of Fair/Good, GSI value for this case would be between 60-70 with average of 65. Now by using Hoek and Diederichs (2006) equation, deformability modulus of rock mass would be around 10 GPa. As mentioned before, this method does not provide ability to calculate the rock mass modulus at different depths.

**CONCLUSIONS**

Deformability modulus of jointed rocks is a key parameter for all numerical and analytical analysis in rock engineering fields. Induced deformations in intact rock and discontinuities control the deformability modulus. Deformability modulus is a stress dependent parameter and increases as applied stress increases. All well-known empirical formulations do not consider this property of deformability modulus. In this paper, stress dependency of normal and shear stiffness of rock discontinuities were included in an existing analytical formulation. This technique was applied to a real case and it was found that deformability modulus increases significantly with depth increase and for depths higher than 200 m it approaches to the elastic modulus of intact rock. The results of this study highlighted the shortcoming of empirical methods to determine a reliable value for deformability modulus of jointed rocks.

**REFERENCES**


THE EFFECT OF TEMPERATURE ON THE APPLICATION OF CHEMICAL SOLUTIONS TO ROCK

Vivian Yuan and Paul Hagan

ABSTRACT. Certain chemical solutions have been shown to reduce the degradation effect of water on the material properties of rock and increase the strength and cuttability of clay-bearing rock. Consequently, there is potential for the application of chemical solutions to enhance the cutting performance of mechanical excavators and for maintaining ground stability in clay-bearing rocks that exist surrounding coal formations. A study investigated the effect of temperature of chemical solutions on rock properties. The temperatures ranged between 5°C and 40°C. It was found that temperature effect varied with different solutions. In the case of potassium chloride, there were changes in abrasivity, weatherability and cuttability of rock with temperature whereas the results were less consistent with magnesium chloride and copper sulphate solutions.

INTRODUCTION

A challenge encountered by drag pick mechanical excavators is that cutting rates reduce significantly in clay bearing rock due to swelling of clay minerals by in some cases over 50% (Bilgin et al., 2004). In the presence of groundwater, clay softens and consequently reduces its strength and increases its deformability (Brady and Brown 2004). Underground coal mining operations can be adversely affected by clay swelling where the floors of coal seams are comprised of shales with bogging of machinery resulting in productivity losses and unsafe working conditions (Anwar et al., 1998). The slaking and collapsing behaviour of certain clay bearing rocks also reduces the stability of the roof and sidewalls of underground excavations.

Research over the past decade has found that certain chemical solutions can reduce the degradation effects of water, and increase strength and cuttability of clay bearing rock (Morkel and Saydam 2008 and Hagan et al., 2012). Therefore, there is potential for chemical solutions to be used to enhance the cuttability of rock and stabilise excavations. These changes have been attributed to the exchange of cations in solution with those in clay minerals as was demonstrated by a reduction in the cation concentration in solution after treatment of rock (Elias 2010). Summersby (2012), Deramore-Denver (2010) and Elias (2010) reported on studies into the effect of immersion time and concentration on rock properties but there has been very limited research on how the temperature of chemical application affects rock. The only relevant finding in literature was that treating kimberlite with magnesium chloride at 40°C degraded more compared to when it was treated at room temperature (Morkel et al., 2007). It is understood that temperature is a factor that affects the rate of chemical reactions; hence temperature could impact on how quickly solutions will alter rock properties – the extent of which was investigated in this project.

PROJECT OBJECTIVES

The objective of this project was to determine if the temperature of chemical solution application affects various rock properties, namely strength, cuttability and weatherability. The chemical solutions examined in this study were potassium chloride, magnesium chloride and copper sulphate, which have been previously shown to cause significant changes to these rock properties. As the temperature of Australian underground mines varies considerably up to 40°C (Hassell et al., 2004), the temperature range investigated in this study was between 5°C and 40°C.
EXPERIMENTAL PROCEDURE

Sample preparation
Test specimens of Hawkesbury Sandstone were diamond cored from blocks sourced from Gosford Quarries on the Central Coast, NSW, and specimens of Althorpe Claystone were sourced from Bulga Underground Mine near Singleton, NSW. X-ray powder Diffraction (XRD) was used to identify the clay mineral composition of the rocks. The Hawkesbury Sandstone specimens contained 7% illite and 6% kaolinite, whilst Althorpe Claystone contained 20% illite and 3% palygorskite.

UCS, Brazilian tensile strength and core cuttability tests were performed on Hawkesbury Sandstone, whilst the slake durability test was conducted using Althorpe Claystone. The CERCHAR abrasivity test was carried out on both types of rocks. Different 1.5 mol solutions were prepared by dissolving potassium chloride, magnesium chloride and copper sulphate salts in water. This concentration was selected due to its proven effectiveness and to allow for comparison with past studies.

The core specimens were completely immersed in the chemical solutions as well as in pure water within a sealed container for a set amount of time. The container was placed in a temperature-controlled environment with the exception of the samples treated at room temperature. The 5°C environment was simulated in the laboratory’s industrial fridge, and the 30°C and 40°C environments were replicated in an industrial oven. The rocks treated at ambient room temperature in the laboratory were kept at a relatively constant temperature of around 20°C with thermometer readings ranging between 17°C and 22°C across the treatment period.

The treatment time was also controlled across the specimens for each test. The UCS, Brazilian tensile strength, CERCHAR and slake durability test specimens were immersed overnight for 17 hours, whilst the Core Cuttability test specimens were treated for six hours. The water-only treated specimens were also immersed for these periods, but only at room temperature. Following treatment, all test specimens were air-dried for an hour then wrapped in plastic film until testing.

Core cuttability test
The Invicta 6M linear rock cutting machine fitted with a tri-axial dynamometer, located in the UNSW Mining Engineering Rock Mechanics Laboratory, was utilised for the core cuttability test. The NQ (63 mm diameter) sandstone core specimens were cut to approximately 230 mm in length.

The mass and dimensions of the cores were recorded to determine the density prior to testing. Each core was secured in place in a vice and levelled so that a tungsten carbide pick would run along the axis of the core maintaining a constant depth of cut as shown in Figure 1. The depth of cut in each test remained unchanged at 5 mm. The total length cut, retained length and mass of rock cuttings mass were each measured and recorded before the core was repositioned for the second and subsequent cuts up to four cuts in total for each core. Between each cut, the core was rotated 180°, 90° and 270° from the first position. If the core broke during a cut, the following cut was performed using the largest remaining portion of core specimen.

Figure 1: Cut performed along the longitudinal axis of sandstone core
The triaxial dynamometer measured the transient changes in the directional strain during each test at a rate of $1 \times 10^3$ samples per second and the values stored on a data acquisition system. The carbide loss was determined by recording the mass of the carbide before and after each set of four cuts on a core specimen.

**Uniaxial compressive strength test**

The MTS 815 machine was used to conduct the UCS test. The test was carried out using sandstone specimens of approximately 57 mm diameter and 149 mm in length with a height to diameter ratio of 2.61. Three specimens were prepared and tested for each combination of chemical solution and temperature.

A constant force rate of 0.002 kN/s was applied to each specimen until failure (ASTM, 2010). The data acquisition system generated a plot of the force versus time and the peak force value was used to calculate the UCS if the sample had exhibited shear failure, which was the case for all but two samples.

**Brazilian tensile strength test**

The Brazilian or indirect tensile test was also performed using the MTS 815 machine. Sandstone specimens of approximately 57 mm diameter and 20 mm thickness were used for this test. Four specimens were prepared and tested for each combination of chemical solution and temperature.

In this case a constant force at a rate of 0.002 kN/s was applied (ASTM 2008). The data acquisition system generated a plot of the force versus time and the peak force value was used to calculate the indirect tensile strength. In this test, failure occurs when diametric cracks form between the platen contact points.

**CERCHAR abrasivity test**

The CERCHAR abrasivity test was performed in the method suggested by Alber et al., (2014) using the West (1989) design of testing apparatus. The test arrangement consists of a 7 kg vertical load applied to a sharpened steel pin applied to the top surface of the test specimen secured within a clamping vice. A dial gauge is used to measure the distance as the pin is made to traverse 10 mm across the rock surface as show in Figure 2. A microscope was then used to measure the length of resultant wear flat on the surface of the pin to an accuracy of $\pm 0.01$ mm. The test is replicated for each test specimen five times using five different pins. The sandstone and claystone specimens were approximately 60 mm in diameter and 50 mm in height. One specimen was tested for each combination of rock type, chemical solution and temperature.

![Figure 2: Pin scratching surface of CERCHAR specimen](image)

**Slake durability test**

This test was carried out using the slake durability device with steel drums comprised of 2 mm mesh and plastic troughs for holding water. Each set of specimens consisted of ten lumps of claystone weighing between 40 and 60 g, to give a total mass of between 450 and 550 g. Care was taken to randomly select
sets of rock samples that represented the variability of the Althorpe Claystone. One set of specimens was tested for every rock type, chemical solution and temperature. The mass of the steel drums with and without the claystone were measured before the test. The samples were dried overnight in the steel drums for 17 hours at 85°C, cooled at room temperature for 30 minutes and weighed before each cycle in the slake durability device. Each cycle involved rotating the steel drum in a water trough for 10 minutes at 20 rpm to expose the claystone to wetting and abrasion. Four cycles were performed to be consistent with the experiments of Summersby (2012), Deramore-Denver (2010), Elias (2010), and Morkel and Saydam (2008).

RESULTS AND ANALYSIS

Core cuttability test

Table 1 shows a summary of the core cuttability test results.

Table 1: Summary of core cuttability test results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cutting Force (kN)</th>
<th>Normal Force (kN)</th>
<th>Yield (m³/km)</th>
<th>Specific Energy (MJ/m³)</th>
<th>Carbide Wear (mg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>1.215</td>
<td>0.938</td>
<td>0.0764</td>
<td>15.95</td>
<td>2.468</td>
</tr>
<tr>
<td>Water Treated</td>
<td>1.011</td>
<td>0.751</td>
<td>0.0661</td>
<td>15.33</td>
<td>2.481</td>
</tr>
<tr>
<td>KCl 5°C</td>
<td>1.135</td>
<td>0.912</td>
<td>0.0739</td>
<td>15.42</td>
<td>1.308</td>
</tr>
<tr>
<td>KCl 20°C</td>
<td>1.123</td>
<td>0.910</td>
<td>0.0717</td>
<td>15.63</td>
<td>1.070</td>
</tr>
<tr>
<td>KCl 30°C</td>
<td>1.079</td>
<td>0.715</td>
<td>0.0697</td>
<td>15.57</td>
<td>1.510</td>
</tr>
<tr>
<td>KCl 40°C</td>
<td>0.887</td>
<td>0.597</td>
<td>0.0687</td>
<td>13.04</td>
<td>2.719</td>
</tr>
<tr>
<td>MgCl₂ 5°C</td>
<td>1.031</td>
<td>0.795</td>
<td>0.0719</td>
<td>14.48</td>
<td>1.707</td>
</tr>
<tr>
<td>MgCl₂ 40°C</td>
<td>0.964</td>
<td>0.697</td>
<td>0.0743</td>
<td>13.00</td>
<td>2.025</td>
</tr>
<tr>
<td>CuSO₄ 5°C</td>
<td>1.014</td>
<td>0.738</td>
<td>0.0726</td>
<td>14.07</td>
<td>2.876</td>
</tr>
<tr>
<td>CuSO₄ 40°C</td>
<td>1.133</td>
<td>0.927</td>
<td>0.0681</td>
<td>16.65</td>
<td>2.108</td>
</tr>
</tbody>
</table>

Cutting and normal forces

The average cutting and normal forces for each chemical and the temperature along with the untreated and water treated controls are displayed in Figures 3 and 4 respectively. The percentage values shown in the following graphs indicate the measured parameter relative to the value of the water treated sandstone core. The error bars represent the variability with each set of test results.

Figure 3: Core cuttability test results for cutting force
While the forces do not vary significantly for the MgCl$_2$ treated samples, the difference between the forces for the CuSO$_4$ treated test specimens may be due to the lower concentration of the chemical at 5°C. At 5°C, the solubility of the chemical is between 231 and 275 g per litre (Dean and Lange 1999) which limited the concentration of the solution to 1 mol. It was observed that some of the CuSO$_4$ in solution had crystallised after the immersed samples were removed from the industrial fridge. Therefore, it is likely that rock samples treated with copper sulphate at 5°C were exposed to a concentration lower than 1.5 M.

From Figures 3 and 4, it can be observed that the cutting and normal forces for the KCl treated sandstone are similar at 5°C and 20°C, but that tend to decrease as temperature rises from 20°C to 40°C. These changes with temperature are plotted in Figure 5 showing the relationship between forces and temperature of KCl application can be represented by a bi-linear function.
Yield and specific energy

Figure 6 shows that yield slightly increased across the full range of chemical solution temperatures though generally, yield was greatest at the lowest temperatures. Figure 7 shows that there was a slight effect of chemical solution and temperature on specific energy and hence the efficiency of the cutting process though the results were not generally consistent.

There was a substantial decrease in the specific energy for the KCl 40°C samples, mainly due to the low cutting force. The difference in the specific energy for the CuSO₄ samples may also be attributed to the lower concentration of CuSO₄ at 5°C. The specific energy for both the low and high temperature MgCl₂ solutions.

Impact abrasivity

There is some variability in the results obtained for impact abrasivity as shown in Figure 8. This was determined from the difference in the carbide mass after the set of cutting tests for each specimen. There was only one data point for each treatment type with KCl 40°C being the one exception with two results. Therefore, it is recommended to carry out more experiments to allow for conclusions to about temperature to be drawn from this test.
Uniaxial compressive strength test

In Figure 9, the results show that UCS is mostly unaffected by the temperature of chemical application with differences of 7% or less relative to water treated samples.

The difference in results may be due to variation of the control variables between the tests, such as rock type, solution concentration and drying time. Deramore Denver (2010) and Elias (2010) performed UCS tests on siltstones sourced from Ravensworth North and Crinum Mines, whereas the results in Figure 9 are for sandstone from Gosford Quarry. The clay mineral constituents identified using XRD analyses are different for the two rock types with nacrite found in the siltstones, whilst the Gosford Sandstone consisted of kaolinite and illite. Although nacrite is polymorphous with kaolinite (Ruiz Cruz 2007) and both clay minerals are in the kaolin clay group (Velde 2012), comparison of the results show that KCl and CuSO4 may not have the same effect on the two different clay minerals. The presence of illite in the sandstone also limits the comparability of the results.

Morkel and Saydam (2008) found that submerging kimberlite specimens in 1.5 M KCl for 30 min before drying for 18 h resulted in UCS increasing above the value for water treated specimens. Aside from the
difference in treatment and drying times, the kimberlite contains smectite, which is a group of clay minerals known for their swelling behaviour. The contrast in results suggest that treatment and drying time have a significant impact on UCS and that KCl may be more effective on certain clay minerals than others.

**Brazilian tensile strength test**

The results (Figure 10) show that Brazilian tensile strength is largely unaffected by the temperature of KCl and MgCl₂ application with differences of 11% or less relative to water treated samples. The Brazilian tensile strength for sandstone treated at 5°C and 40°C differed by 26%, which may also be due to the lower concentration of CuSO₄ at 5°C.

![Brazilian Tensile Strength Test Results](image)

**Figure 10: Brazilian tensile strength test results**

The chemically treated samples all yielded higher Brazilian tensile strengths compared to the water treated samples with KCl being the most effective; improving strengths to approximately the same level as untreated samples.

**CERCHAR abrasivity test**

**Sandstone**

The results in Figure 11 show that the CERCHAR Abrasivity Index (CAI) for the chemically treated samples were all significantly lower than both the untreated and water-treated specimens, which suggests that the application of KCl, MgCl₂ and CuSO₄ to sandstone reduces its abrasivity. This finding is consistent with most of the impact abrasivity results from the core cuttability test, although more core cutting experiments are recommended to improve the reliability of the test results. The reduction in the abrasivity of rock due to treatment with KCl, MgCl₂ and CuSO₄ to sandstone reduces its abrasivity. This finding is consistent with most of the impact abrasivity results from the core cuttability test, although more core cutting experiments are recommended to improve the reliability of the test results. The reduction in the abrasivity of rock due to treatment with KCl, MgCl₂ and CuSO₄ to sandstone reduces its abrasivity. This finding is consistent with most of the impact abrasivity results from the core cuttability test, although more core cutting experiments are recommended to improve the reliability of the test results. The reduction in the abrasivity of rock due to treatment with KCl, MgCl₂ and CuSO₄ to sandstone reduces its abrasivity. This finding is consistent with most of the impact abrasivity results from the core cuttability test, although more core cutting experiments are recommended to improve the reliability of the test results.

**Claystone**

There was a significant reduction in the CERCHAR abrasivity of the claystone with chemical treatment in comparison to the plain water-treated specimen as shown in Figure 12. Interestingly the reduction was less significant when compared to the untreated specimen. Though the trends were less apparent, it would seem that a reduction in solution temperature enhanced the effect of the chemical treatment in terms of reduction in abrasivity.

Whilst the CAI did not vary significantly for claystone treated with MgCl₂ at 5°C and 40°C, it differed by 22% relative to the untreated sample for samples treated with CuSO₄ at the two temperatures. This difference may also be due to the lower concentration of CuSO₄ at 5°C.
It was demonstrated that the application of KCl, MgCl$_2$ and CuSO$_4$ to sandstone reduced its abrasivity relative to untreated and water treated samples. Figure 12 shows that this phenomenon is also generally true for claystone, with the exception of the KCl 40°C and CuSO$_4$ 40°C samples where the CAI values are only marginally greater than the untreated sample. Further CERCHAR abrasivity testing for claystone should be undertaken to improve confidence in these results.

**Slake durability test**

The Slake Durability Index (SDI) test results are provided in Table 2 and are plotted in Figure 13 showing the differences in the degradation rates.

**Table 2: Results for the Slake durability index test**

<table>
<thead>
<tr>
<th>Chemical solution</th>
<th>Temp (°C)</th>
<th>$I_{d1}$ (%)</th>
<th>$I_{d2}$ (%)</th>
<th>$I_{d3}$ (%)</th>
<th>$I_{d4}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KCl</td>
<td>5</td>
<td>98.34</td>
<td>97.70</td>
<td>97.00</td>
<td>96.39</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>98.22</td>
<td>97.62</td>
<td>97.12</td>
<td>96.71</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>98.56</td>
<td>97.80</td>
<td>97.20</td>
<td>96.79</td>
</tr>
<tr>
<td>MgCl$_2$</td>
<td>5</td>
<td>98.09</td>
<td>97.44</td>
<td>96.88</td>
<td>96.47</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>97.48</td>
<td>96.63</td>
<td>95.82</td>
<td>95.32</td>
</tr>
<tr>
<td>CuSO$_4$</td>
<td>5</td>
<td>96.16</td>
<td>94.94</td>
<td>94.11</td>
<td>93.51</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>97.29</td>
<td>96.38</td>
<td>95.67</td>
<td>95.25</td>
</tr>
<tr>
<td>Untreated</td>
<td></td>
<td>96.98</td>
<td>96.04</td>
<td>95.53</td>
<td>95.20</td>
</tr>
</tbody>
</table>
Figure 13: Slake durability test results

It was observed that the claystone specimens treated with KCl over the range of temperatures from 5°C to 40°C performed consistently irrespective of the treatment temperature. The SDI of the KCl treated samples was very similar for all four cycles. However, there is a trend for the SDI to increase slightly as the KCl treatment temperature increases from 5°C to 40°C. The SDI of the KCl treated samples were significantly higher than the SDI of the untreated sample, which confirms that KCl is a strong degradation inhibitor for claystone irrespective of the temperature of the treatment solution.

The SDI for MgCl₂ and CuSO₄ treated samples differed significantly depending on whether treatment temperature was 5°C or 40°C. When treated with these two chemicals at 40°C, the SDI results were very similar to that of untreated claystone. However, when treated with MgCl₂ at 5°C, the SDI is similar to that of the KCl samples. Therefore, the effectiveness of MgCl₂ as a weathering inhibitor depends on temperature of application with 5°C being more effective than 40°C. These results align with the finding of Morkel et al., (2007) that disintegration of kimberlite increases by 27% when treated with MgCl₂ at 40°C compared to MgCl₂ at room temperature (~20°C).

The CuSO₄ 5°C sample had a considerably lower SDI than the CuSO₄ 40°C sample, which indicates that the application of CuSO₄ at the lower temperature increases degradation of claystone. Further testing should be carried out for claystone with MgCl₂ and CuSO₄ applied at temperatures between 5°C and 40°C to determine the effect of treatment temperature between these two points.

Summersby (2012) and Morkel et al., (2007) had found that cations have a strong effect on the weatherability of two types of clay bearing rock, kimberlite and Ulan Claystone, with the extent of weatherability increasing in the order: Cu²⁺ > Mg²⁺ > K⁺. The slake durability test results in this study confirm that this trend is also exhibited in the Althorpe Claystone.

CONCLUSIONS

In general it was found that aside from the few exceptions noted below, the temperature at which KCl, MgCl₂ and CuSO₄ solutions are applied to rock did not affect cuttability, UCS, Brazilian tensile strength, abrasivity and weatherability.

The first exception is that the cutting and normal forces for sandstone decreased linearly as the temperature of KCl treatment increased from 20°C to 40°C.

Second, the CERCHAR abrasivity of sandstone and the weatherability of claystone also increased slightly as the temperature of KCl treatment increased from 5°C to 40°C.

Third, the temperature of the chemical solution significantly affects the weatherability of MgCl₂ and CuSO₄ treated claystone. When claystone was treated with either of these chemicals at 40°C, there were no changes in weatherability relative to untreated claystone. However, when treated with MgCl₂ at 5°C, the slake durability of the claystone improved to a similar level to that of the KCl treated samples. By contrast, the degradation of claystone increased substantially when treated with CuSO₄ at 40°C.
While the Slake Durability test results confirmed the findings of Summersby (2012) and Morkel et al. (2007), which was that cations have a strong effect on the weatherability of clay bearing rock, with the extent of weatherability increasing in the order $\text{Cu}^{2+} > \text{Mg}^{2+} > \text{K}^+$, the UCS and core cuttability results were not consistent with past literature. The UCS of the chemically treated samples was similar to that of the water treated samples, rather than higher in comparison as demonstrated by Elias (2010), Deramore-Denver (2010), and Morkel and Saydam (2008). Also, the cutting and normal force results in this study do not show that forces of the chemically treated samples are lower than the water treated sample as demonstrated by Summersby (2012) and Deramore-Denver (2010). The difference in results may be due to variation of the control variables between the tests, such as the type of clay mineral within the rock and drying time.

It was found that all three chemical solutions examined in this study reduced the abrasivity of rock when compared with water treated samples. The effect was most pronounced with the CERCHAR results for sandstone, which shows that the chemicals perform similarly to reduce the CAI by 36% relative to untreated sandstone.

Also, the chemically treated samples all had higher Brazilian tensile strengths compared to the water treated samples with KCl being the most effective, improving strengths to approximately the same level as untreated samples.

ACKNOWLEDGEMENTS

The author acknowledges the support of the following people for the completion of this research project: Kanchana Gamage for his practical advice and assistance in the execution of the experimental component and Brad Person for allowing access to the claystone core samples. The first author also wishes to thank her family and friends for their support and encouragement.

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GEOTECHNICAL ASPECTS OF PLACE CHANGE MINING AT ENSHAM

Nick Gordon

ABSTRACT: The Ensham bord and pillar mine in the central part of the Bowen Basin is developing panels in the 5-6 m thick Aries/Castor Seam, using the place change mining method. The development mining height is typically 3.5 m, with the remaining floor coal and bell outs extracted on retreat. The depth of cover ranges from 45 m up to 150 m. In geologically unstructured areas, low-density roof support patterns are installed due to the competent roof conditions provided by the high ash, roof coal.

Ensham is currently producing at an average rate of 0.96 Mtpa from each of the three development units. To achieve these levels of productivity, optimisation of the geotechnical design aspects of the mine including the ground support, pillar dimensions, panel orientation, selection of mining horizon, mining at shallow depths of cover and drivage through fault zones has been necessary. This is supplemented with regular inspections and ongoing assessment of the geological and geotechnical conditions, as development progresses into new mining areas.

INTRODUCTION

The Ensham Resources Pty Ltd (ERPL) open cut and underground operation mines seams from the Rangal Coal Measures, in the central part of the Bowen Basin (Figure 1). The open cut operation started in 1993 but with increasing depth the productivity of the operation has decreased and the decision to access the large underground resource was made.

Excavation of the travel and belt road drifts from the open cut, backfilled to the Aries 1 (A1) Seam level, commenced in March 2011 to access the underground reserves (Figure 2). These drifts are inclined at 1 in 8 and reached the coalesced Aries 2/Castor (A2C) Seam in September 2011, approximately 200 m from the portal entries.

Figure 1: Location Plan

Principal Geotechnical Engineer, Gordon Geotechniques Pty Ltd. ggpl@hotkey.net.au, M: 0428 186255,
The underground workings at Ensham are characterised by typically good roof and rib conditions at relatively shallow depths, with no gas makes detected from the Aries/Castor Seam. As shown in Figure 2, the panels are laid out between a number of geological faults, with the pillars dimensioned to satisfy the mine’s consent conditions below the Nogoa River and flood plain. At this stage, mining has been carried out in several panels below the Anabranch channel but has not been continued under the Nogoa River (Figure 2).
ENGINEERING GEOLOGY

Seam Characteristics

The underground workings are located in the central part of the Ensham mining lease where the Aries/Castor Seam (A2C and A3C) is typically 5-6 m thick, at 45-150 m depth of cover (Figure 3 and Figure 4). The maximum depth in the western part of the planned workings is 200 m (Figure 3).

To the north and south of the underground workings, a number of roof and floor splits are present in the Ensham area (Figure 4). The most significant split that has affected mining operations to date is the A211 roof seam split (Figure 5). Where this roof split is typically closer than 3 m above the main seam, tensile cracking of the coal roof has been experienced, at depths of 80-100 m.
Roof and Floor Lithology

The immediate stone roof above the Aries/Castor Seam typically consists of interbedded siltstone and sandstone, with occasional thicker sandstone units <1 m thick (Figure 6). The immediate 100-200 mm of roof is weaker, averaging 10-20 MPa and may delaminate when mining to stone roof. The average strength of the immediate 2 m of stone roof is 20-40 MPa, increasing to 30-40 MPa for the 2-6 m horizon.
The immediate stone floor consists of mudstone, grading to siltstone then sandstone, with an average strength of 10-15 MPa. Very weak floor layers (<2 MPa), which could potentially lead to pillar punching and significant failure of the floor have not been identified in the current mining area. Minor heave (<100 mm) of the basal coal plies left after floor coaling has been observed in localised parts of the secondary workings. These observations are consistent with the Floor Strength Index (FSI)\(^1\) values greater than 4 in the majority of the area. Experience at other Bowen Basin mines indicates significant floor heave is more likely to occur when the FSI is <3.

**Faults**

3D Seismic has been carried out over the majority of the Ensham underground area to supplement the existing close spaced exploration borehole data and identify geological features ahead of mining (Figure 7). The seismic has been found to locate the faults and even rolls to an accuracy of 10-20 m. This has allowed the layout of the panels to be optimised between the geological features (Figure 2). From a safety perspective for operational personnel, the accuracy of the seismic also allows standoff zones to be put in place as development advances towards predicted faults.

![Figure 7: 3D Seismic Coverage and Exploration Borehole Data](image)

The major faults (throws up to 12 m) encountered in the underground workings so far have been normal in character and relatively clean (Figure 8). The fault zones are also associated with greasy backs,

\[ \text{Strength Index} = \frac{\text{Strength}}{(\text{Depth} \times 0.025)} \]

\(^1\) Strength Index = Strength/(Depth*0.025)
slickensides and smaller faults, which may have reverse characteristics. Delamination of the coal roof may also occur in close proximity to the faults.

![Figure 8: Clean Normal Faults](image)

**Horizontal Stress Direction**

The occurrence of roof guttering in the immediate 100-200 mm of weaker stone roof has only been encountered in geologically structured areas and is consistent with the NE orientation of the major horizontal stress determined by acoustic scanner and overcoring (Figure 9). The guttering has not been observed to progress higher up into the roof.

![Figure 9: Examples of Guttering of Weak Immediate Roof Stone](image)

Where tensile cracking of the roof coal has been encountered, it is inferred that there has been compressive failure of the weak stone interburden between the A211 seam split and the main seam. This cracking is restricted to areas where the Roof Strength Index of the stone is less than about 6 (Gordon and Tembo, 2005).

**MINING METHODOLOGY**

The development of bord and pillar panels at Ensham uses the place change methodology in two stages:

- The primary development height is typically at 3.5 m, leaving high ash coal in the roof to prevent dilution and provide a competent roof beam.
- Floor coal (typically 1-1.5 m thick) and bell outs are mined on retreat.
Development

Development roadways are nominally 6.5 m wide and 3.5 m high, excavated with double pass, 3.6 m narrow head Joy 12CM27 continuous miners. The coal is loaded into Joy 10SC32D shuttle cars and the plunges are bolted with Joy multibolters. The roadway height is decreased to 3.2-3.3 m in thinner seam areas to improve wheeling conditions. Poor floor conditions can develop where the coal floor thickness reduces to typically <1 m, due to the continual shuttle car wheeling, particularly in the presence of water.

The maximum plunge distance is 14 m corresponding with the position of the shuttle car driver under supported roof. In poorer ground conditions, the plunge distance is reduced, as well as the number of places that can be left unsupported. In the majority of the workings developed at depths up to 150 m, the typical ground conditions experienced are shown in Figure 10.

Localised poorer ground conditions have been encountered in some parts of the workings including:

- A zone of blocky and cleated roof coal in the NW Mains and at the outbye ends of 101, 102 and 103 Panels (Figure 2).
- Tensile cracking of the coal roof in the A211 seam roof split area, at depths of 80-100 m.
- Slabbing of the immediate 100-200 mm of weaker stone roof.
- Shallow delamination of the roof (both coal and stone) in unsupported plunges driven towards geological structures.

![Figure 10: Typical Development Conditions](image)

A key aspect of productive place change mining is the management of water. The excavation of floor sumps in the flanking headings is routinely used to catch water and keep it away from the face area.

Bottom coaling and mining of Bell Outs

On retreat, a 5.5 m wide head continuous miner is used to mine the floor coal and extract the bell outs on the outside of the panel. This wide head miner leaves a nominal 0.5 m canche on each side of the roadway, which has a number of benefits including (Figure 11):

- Reduce rib bolt requirements.
- Improve pillar stability.
- Reduce the amount of tramp and rubbish that is loaded out.
The bell out sequences typically contain 1500-2000 tonnes of coal in non-structured conditions, where 14 m plunges are stable (Figure 12). In structured areas, the plunge distance is decreased and stooks are left to reduce the unsupported span.

By developing stubs of consistent length on both sides of the panel and maintaining a standard sequence of extraction for the bell outs, a pillar is formed with dimensions similar to the panel pillars (Figure 12). At the inbye end of the bell out entry stub, an infill row of breaker line roof bolts is also installed. In poorer ground conditions, such as in close proximity to geological features and/or within the seam split zone, these infill bolts are supplemented with additional roof support.

Initially, the SW Mains Panel was developed with nine headings but operational personnel have since found that the drivage of seven headings with stubs driven on the flanks of the panels to allow the mining of bell outs on retreat is more productive (Figure 2). For a seven heading panel layout with bottoms extraction and bell out mining on retreat, 60% of the coal is produced without the requirement for bolting.
Production Rates

Ensham is currently producing at an average rate of 0.96 Mtpa per unit and produced 2.62 Mt in 2015. Peak rates have reached 1.15 Mtpa per unit. The best production in a month of 305,000 tonnes was achieved in December 2015 and 200,000 t/month has been consistently produced since May 2015 (Figure 13). Average rates in development are around 230 t/operating hour and 210 t/operating hour when mining floor coal and extracting bell outs. These ERPL unit rates are comparable with the top producing bord and pillar operations in both the USA and South Africa.

![Figure 13: Production Rates](image)

A fourth floor brushing contract unit commenced in late October 2015 with a wide head continuous miner, to supplement the development coal produced by the Ensham narrow head, double pass continuous miners.

MINING HORIZON

The pit bottom roadways and the initial mains drivage were mined to stone roof; however, delamination of the immediate 100-200 mm of weaker roof occurred in some plunges, requiring the routine application of mesh (Figure 14).

Because of this delamination, mining to a high ash coal roof was trialled in the SW Mains and was found to provide a more competent roof beam (Figure 10). Leaving the high ash coal in the roof also improved the quality of product, as the coal is simply crushed and screened before loading onto the trains. It should be highlighted that the coal roof beam does not perform as well in geologically structured areas (Figure 14).

The miner drivers have utilised the penny band, located typically 0.8-1.2 m from the top of the seam, for horizon control. In most areas, this band is a distinct off white colour and can be easily followed by the operators (Figure 15). Towards the north, the band does becomes darker and slightly harder to follow in places.
Due to the variability in the seam thickness across the underground mining area, the amount of coal left in the roof varies from 0.5-0.6 m in the thinner seam areas, up to 0.9 m in thicker seam panels such as 103. These roof coal thicknesses are consistent with a voussoir beam analysis of high ash Aries/Castor Seam roof coal, spanning across a 6.5 m wide roadway (GGPL 2015).

In the zone of blocky and jointed coal roof encountered at the outbye ends of 101-103 Panels, the roadways were mined to the base of the 0.3 m thick A211 ply at the top of the seam (Figure 16). As detailed in the Development TARP, roof meshing is required when mining to this horizon due to the potential for the delamination of the coal plies.

Figure 14: Delamination of both Stone and Coal Roof

Figure 15: Penny Band
PILLAR DESIGN

Pillar design at Ensham uses the empirical methodology developed by the UNSW (Galvin et al, 1998). The standard pillar sizes at shallow depths of cover (<125 m) are 21 m x 28 m (centre) and 22.5 m x 28 m (centre) pillars. Where the depth increases to >125 m the minimum centre distance is increased to 24 m. Larger wings are formed in the headings to allow the cut-throughs of the 24 m pillars to be holed in one sequence.

As the size of the underground workings increases, the importance of leaving suitably sized barriers between panels cannot be emphasised enough. Furthermore, with increasing depth either compartmentalisation of extraction areas using intra-panel barriers or reduction in floor coal thickness is planned to ensure long term stability below the flood plain (Mine Advice Pty Ltd 2015). In addition, the location of geological features is considered when designing the floor coaling sequence. As part of the mine’s consent conditions, all coal pillars beneath the Nogoa River flood plain must have a FOS>1.6.

Now that a number of panels have been developed at Ensham and bottom coal extracted, a back analysis of the pillar dimensions in each panel so far (maximum heights used) is presented below in Figure 17. This figure shows that the Ensham pillars mined in the shallower part of the area, plot well away from the empirical database of Australian and South African failed pillars presented by Hill (2005).

The pillar design at Ensham has also considered the strength of the floor. Bearing capacity analysis indicates that floor strengths of <2 MPa are required before failure can occur below 21 m x 28 m (centre) pillars. Analysis of the sonic velocity values for the immediate floor across the area indicate typical average floor strengths >10 MPa.

GROUND SUPPORT

The Code Green development ground support consists of 4 x 1.5 m roof bolts every 1.5 m in a 6.5 m wide roadway, supplemented with 1 x 1.5 m rib bolt mid pillar and 2 x 1.5 m rib bolts on the corners. The AX roof bolts are anchored with 500 mm fast set resins providing an average encapsulation of 927 mm (87 measurements) when installed in 28 mm diameter holes. The roof bolt length is increased in seam split and faulted conditions.
In coal roof areas, meshing is only carried out in the services roadway (a sheet every second row of bolts) or when triggered by the TARP (Figure 18). In stone roof areas, all roadways are meshed due to the potential for delamination of the immediate 100-200 mm of weaker stone roof (Figure 14).

![Diagram of roof stability analysis](image)

**Figure 17:** Pillar factor of safety and width: height ratio.

**Figure 18:** Roof mesh in the services road installed every second row of bolts

Pull testing is routinely carried out at Ensham, with all roof bolts pulled to >13 tonnes without movement (105 tests). Additional short encapsulation tests have been carried out in both coal and stone, indicating that the anchorage in the high ash roof coal is comparable to the overlying immediate roof stone (Figure 19).
When installing 1.5 m roof bolts, 1.5 m rib bolts are correspondingly used to simplify the bolting and supply process. Analysis of potential wedges indicates that these 1.5 m rib bolts can prevent sliding on cleat planes inclined at <70° in 4 m high roadways. The inclination of the cleat in the workings is typically >80°, indicating sufficient anchorage is available behind potential failure planes in the rib. 103 Panel has now been developed to 140 m, with only minor spall encountered on corners less favourably aligned to the dominant north/south cleat direction.

The typically good ground conditions in the underground workings are confirmed by the CLOCKIT readings, with the majority of movement less than the Code Blue trigger of 10 mm (Figure 20). In the A211 seam split zone, where tensile cracking was encountered, approximately 60% of the CLOCKITs show movement between 4-10 mm. This is compared to outside the seam split area, where 90% of the movement measured is <2 mm.
MINING AT SHALLOW DEPTH

Development was carried out at the inbye end of 203 Panel where the depth of cover reduced to 45 m, with 27 m of fresh rock head (Figure 3). The immediate roof strength in this area was as low as 6-8 MPa, however stable 14 m plunges were still achievable on development, followed by floor coaling and mining of bell outs.

Experience at other mines indicates potholes due to tensile failure of intersections generally reach the surface at depths <50 m. As such, an assessment of the risk of pothole subsidence was carried out, prior to development in 203 Panel, using the limiting equilibrium analysis documented in Brady and Brown (2006). Both dry and wet conditions were analysed, with failure along cohesionless joints assumed. The stress ratio was also reduced to account for the shallow depth of cover.

To maintain a Factor of Safety of >2 in wet conditions, at least 20 m of fresh rock is required for a large intersection with side dimensions of 10 m, applying a Factor of Safety of 2 (Figure 21). This analysis is consistent with the good conditions experienced at the inbye end of 203 Panel.

CONCLUSIONS

To maintain the current production rates in the place change operation at Ensham requires regular inspections and ongoing assessment of the geological and geotechnical conditions, supplemented with back analysis. This assists in fine tuning the strata control aspects of the mining process, as development progresses into new mining areas.

A key element in the continuity of the operation is the accuracy of the 3D seismic, which supplements the exploration borehole data. Not only does this allow panel geometries to be optimised, it also provides proactive warning of deteriorating ground conditions to the underground workforce. Ensham plan to continue this exploration approach in new mining areas.
ACKNOWLEDGEMENTS

The author acknowledges Ensham Resources for allowing presentation and publication of this paper. Special thanks to operational personnel including John Hart (Mine Manager), Christian Ecker (Production Superintendent) and Peter Liston (Technical Services Superintendent), as well as the underground operators, who have provided invaluable feedback on the conditions encountered at Ensham.

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INVESTIGATION INTO ROOF SUPPORT BEHAVIOUR AT GRASSTREE MINE

Terry Medhurst¹, Jason Emery² and Adam Huey³

ABSTRACT: The results of a recent investigation into roadway roof support behaviour using a Geophysical Strata Rating (GSR) based support design method are presented. This paper outlines details of a study of roof support behaviour at two instrumented sites at Grasstree Mine. The impact of differing support elements were investigated and tested against developments in roof beam analysis in order to optimise the support process.

INTRODUCTION

Modern methods of strata characterisation and modelling such as the Geophysical Strata Rating (GSR) have been successfully developed and implemented 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 suggest a similar principle may apply. Initial studies demonstrated the capabilities of a newly developed analytical model to quantify the relationship between support practice and roof convergence (Medhurst 2015).

The role of stress and the impacts of weaker roof are increasingly impacting on development rates and productive capability particularly at those sites nearing the end of mine life. The industry is therefore requiring site based tools to help drive change processes, aid decision making and improve Trigger Action Response Plans (TARPs). In this context it was proposed to test the capabilities of the new design approach as part of a broader program to optimise roof support at Grasstree Mine. A detailed geotechnical study was initiated that included comprehensive instrumentation and monitoring of a roadway in conjunction with trials of new ground support hardware.

ROOF CONDITIONS AT GRASSTREE

Figure 1 shows the level of convergence based on 4-anchor mechanical extensometer (Tell Tale) data along the Main headings at Grasstree Mine. A marked increase in roof convergence past 40 ct is evident in the plot. Roof support consisted of 6 x 1.8 m bolts at 1.3m spacing in the headings, with 2 x 6m point anchored Superstrands every 2.6m across the intersections up to 50 ct. This support density was generally adequate up to 40ct (+300m cover depth) with the exception of a few isolated areas whilst a four bolt pattern with row spacing up to 1.4 was successfully installed outbye of 30 ct in places (<275 m cover depth). From inbye 40 ct additional guttering was observed in both headings and cut throughs. This correlates with the transition to a thick laminated sandstone/siltstone roof, increasing mica content and cover depth reaching 325 m. A six bolt pattern on 1.3 m spacing became marginal from 45 ct inbye, with excessive remedial support required, particularly across the cut throughs and intersections, due to excessive guttering, bagging and centreline cracking.

Over time the Grasstree roof support system evolved to shorten bolt row spacing to 1m and reduce cable length to 4 m however these are at minimum 75% resin encapsulation. This system was very effective in the 800 s series panels and up to 904 panel on the northern side of the mine. However at cover depth exceeding 350 m (905MG and beyond) this support regime also became marginal in places, even with Superstrands installed at 1 m row spacing. This triggered the installation of post groutable

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cables on advance and also quite often remedial support. Both of these support practices severely impeded development advance rates.

It was decided to find a stiff high capacity cable support that could be installed by hand at the face with a resin anchor overlapping the bolted horizon. This led to the discovery of the Goliath cable. The vision was to not only be able to space the cable supports further apart and achieve the same or better results, but to also potentially install Goliaths at 1 m row spacing replacing the need for post groutable cables. A proposal to undertake a support and instrumentation trial in MG906 was developed.

Figure 1: Roof convergence in Mains at Grasstree Mine

The Geophysical Strata Rating (GSR) system has been used by Grasstree personnel to assist with design and planning since late 2013. This system is now the preferred method for characterising the strata, with particular emphasis placed on support optimisation due to both increasing cover depth and longwall retreat rates. In current development panels GSR for the immediate 3 m of roof typically ranges from 50-65 as shown in Figure 2.

Figure 2: Median GSR over 3m of roof and GSR/σ_H at Grasstree Mine

Figure 2 also shows the GSR to horizontal stress ratio. Previously, a relationship between the onset of roof instability and GSR:σ_H has been established with a GSR:σ_H = 3 being considered a threshold level. Figure 2 also shows the GSR to horizontal stress ratio. Previously, a relationship between the onset of
roof instability and GSR: $\sigma_H$ has been established with a GSR: $\sigma_H = 3$ being considered a threshold level. This ratio is just above that level and a check on those ratios suggests that roadways with a stress concentration factor $> 1.2$ would reach this stress threshold. Increased levels of roof convergence are now being experienced in the deeper parts of the mine leading to an increase in installed roof support density. This suggests that stress related damage is a contributing factor to the observed poor roof conditions.

**MONITORING PROGRAM**

**Geotechnical Setting**

A monitoring site was chosen inbye of 4 ct in the travel road of MG906. Figure 3 shows the monitoring site and the GSR analysis for the borehole (ECC1079) closest to the monitoring locations. Detailed inspection of the GSR data shows that the immediate roof is generally strong and overlain with a weaker siltstone unit. In some areas however the roof is known to be highly anisotropic with weak bedding or micaceous zones present.

![Figure 3: Monitoring location showing cover depth with borehole analysis](image)

Stress measurements were also completed for the study in Hole ECC1073 located just outbye of 4 ct and are summarised in Table 1 (Sigra, 2015). The results suggest a stress ratio in the range $H:V = 1.5$ to $1.9$ in the overlying roof strata.

**Table 1: Stress measurement results**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Location</th>
<th>Material</th>
<th>$\sigma_1$ (°)</th>
<th>$\sigma_1$ (MPa)</th>
<th>$\sigma_2$ (MPa)</th>
<th>$\sigma_V$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>377.0</td>
<td>GC Roof</td>
<td>Siltstone</td>
<td>42.2</td>
<td>15.6</td>
<td>10.3</td>
<td>9.4</td>
</tr>
<tr>
<td>380.0</td>
<td>GC Roof</td>
<td>Sandstone</td>
<td>49.8</td>
<td>14.5</td>
<td>9.5</td>
<td>9.5</td>
</tr>
<tr>
<td>382.96</td>
<td>GC Roof</td>
<td>Siltstone</td>
<td>32.2</td>
<td>17.9</td>
<td>10.7</td>
<td>9.6</td>
</tr>
<tr>
<td>390.02</td>
<td>GC Floor</td>
<td>Siltstone</td>
<td>39.6</td>
<td>11.3</td>
<td>7.9</td>
<td>9.75</td>
</tr>
</tbody>
</table>
Roof Support and Instrumentation

A key aim of the monitoring program was to evaluate roof support performance for typical patterns used at Grasstree, assess the potential for alternative support strategies and devise a method for predicting tolerable convergence based on support density and rock mass competency (GSR). The primary roadway roof support at Grasstree consists of 1.8 m X Grade roof bolts, 4.1m Superstrand cables and either 6.2 m or 8.2 m MW9 Megastrands. One aspect of this study was to investigate the use of Goliath cables in place of Superstrands. A summary of support properties is shown in Table 2 and the instrumentation layout in Figure 4. It should be noted that the Goliaths are comprised of smooth wires and the Superstrands are indented.

Table 2: Support properties

<table>
<thead>
<tr>
<th></th>
<th>X Grade Bolt</th>
<th>Superstrand</th>
<th>Megastrand</th>
<th>Goliath</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>1.8</td>
<td>4.1</td>
<td>6.2/8.2</td>
<td>4.1</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>21.7</td>
<td>21.8</td>
<td>31</td>
<td>28.6</td>
</tr>
<tr>
<td>Hole diameter (mm)</td>
<td>28</td>
<td>28</td>
<td>42</td>
<td>35</td>
</tr>
<tr>
<td>Minimum UTS (t)</td>
<td>30</td>
<td>58</td>
<td>62</td>
<td>99</td>
</tr>
</tbody>
</table>

Additionally, tell-tales were installed at the two sites. Readings from the instrumented bolts were used to determine axial force, axial strain, bending moment and bending strain in the primary support horizon.
Shear strips were installed to gauge the extent of horizontal movement and strain within the supported roof interval and load cells were installed to measure the total load at the base of the tendons. Sonic extensometers were installed to measure the vertical movement within the roof up to 8m above the coal seam and were comprised of 20 anchors. Instrumentation was installed by a combination of Grasstree, SCT and PDR personnel.

**STABILITY ASSESSMENT**

Another objective of the project was to test the capabilities of the roof support analytical model (Medhurst 2015). The two support regimes were therefore assessed prior to the trial in order to evaluate the potential impact of increasing the spacing from 1 m to 2 m using the higher capacity Goliath cables. The analysis shows predicted outcomes based on a range of support patterns using data from Hole ECC1079. The examples are based on a fixed cantilever model. Blue curves apply to initial X Grade bolts installed in roadways. Green and pink curves apply to combined strands and the X Grade bolts in the roadways. The red curves apply to intersections. Figure 5 shows the ground response for the Superstrand pattern and Figure 6 for the Goliath pattern.

In general, replacing Superstrands with Goliaths at 2 m spacing in roadways will produce a similar overall support density, but would reduce the serviceability by about 10 mm. In other words there would be 10 mm less to work with in the TARPs with the Goliath support installed, i.e. note the difference in serviceably limit as predicted by the analytical model. This is the consequence of having a stiffer, higher capacity tendon installed at a wider spacing. In general, it is suggested that provided the beam end constraints are preserved then roof convergence levels would be kept to 30 mm in the roadway regardless of the support type used.

Figure 7 shows an equivalent plot for the Goliaths based on a propped cantilever beam model. Note that roof convergence levels would be predicted to be just over 40 mm at 4 m softening height in the roadway. Note also how the intersection reaches its serviceable limit at about 70 mm at a 6 m softening height. The differences between Figures 6 and 7 show the influence of stress related damage and/or strata relaxation in the roadway.

In general the stability assessment indicated that Superstrands could be swapped out with the Goliaths at the broader spacing, but the roof would be less tolerant to increased levels of roof convergence. In other words, the analysis suggests that the height of softening would reach the same levels of convergence as the Superstrands but with 10 mm less roof movement with the Goliath pattern.

![Figure 5: Ground response for Superstrand pattern](image-url)
As is the case with many underground monitoring programs, problems with data capture and measurements were encountered. In particular, there were issues with the strain bridge monitor used to acquire readings from the instrumented bolts and the shear strips. This led to a large number of missed results, mostly around the centre of the bolts. The shear strips were more resilient due to the use of four separate readings for one strip, however some data was still lost. The sonic extensometer data is shown...
in Figure 8. The results show up to 27 mm of movement on the left hand side and less than 5 mm on the right. Some distinct saw tooth profiles are also present in the data, mostly on the left that are interpreted as the influence of shear movements.

Figure 8: Roof displacements for Superstrand pattern

Figure 9 shows axial force (kN) at Site 1 at 48 days after installation. For the purpose of the contours both rows of bolts (4/2 pattern) have been flattened onto a single plane. The pattern shown here is indicative of all readings at the site, where force is concentrated down the left hand side of the roadway along the stress biased side. Particularly high readings were measured in the out-of-plane (mid-mesh) bolt in the staggered pattern at 0.75 m from the rib. A noticeable bias to the left in roof deformation was also observable in the roadway that is consistent with the measured stress orientation.

Figure 9: Axial force in roof bolts for Superstrand pattern

Figure 10 shows the variation in axial load with time along the staggered bolt. The plot on the left shows axial force at a particular location along the bolt. A consistent increase is observed at the 280 mm location just above the roof horizon, whilst it is relatively constant at the 460 mm level albeit at a higher load. A slow decrease is detected in the upper half of the bolt. The distribution is shown on the right and is plotted relative to time and face position (instrumentation was installed at 73 m). The load increase in the lower 1 m of the bolt over time is evident. The corresponding results from load cell monitoring of the Superstrands is shown in Table 3. The capacity of the load cell (25 t) was reached on the left hand side of the roadway after the face advanced approximately 7m inbye the instrumentation site. Load monitoring in the bolts and Superstrands shows distinct evolution of build-up over the first three days as the face advanced past the “square”, i.e. 5 to 6 m. The shear distribution for the left and right side of the roadway is shown in Figure 11.
Table 3: Load cell results for Superstrand pattern

<table>
<thead>
<tr>
<th>Face Position</th>
<th>LHS (tonnes)</th>
<th>RHS (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Install (73m)</td>
<td>13.8</td>
<td>11.0</td>
</tr>
<tr>
<td>75m</td>
<td>15.1</td>
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</tr>
<tr>
<td>76m</td>
<td>17.50</td>
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</tr>
<tr>
<td>79m</td>
<td>24.8</td>
<td>12.5</td>
</tr>
<tr>
<td>80.5m</td>
<td>~26 *</td>
<td>12.8</td>
</tr>
<tr>
<td>80.5m</td>
<td>~28 *</td>
<td>13.5</td>
</tr>
<tr>
<td>92m</td>
<td>~28 *</td>
<td>14.8</td>
</tr>
<tr>
<td>5 Cut-through</td>
<td>~30 *</td>
<td>15.0</td>
</tr>
</tbody>
</table>

*Load cell maxed out at 25000kPa, estimated from dial position.

A distinct shear zone is present at the 2.5 m to 3 m horizon on the left side, and at 0.95m to 1.35 m on the right with a smaller amount of shear at about 2.5 m. When compared with the sonic extensometer data, the results suggest that horizontal movements up to about 3.5 mm occurred between the 3 m and 2 m horizons, which were measured at about 7 mm in the sonic extensometers. The results indicate that about 50% of the movement detected by the sonic extensometer associated with the shear plane was horizontal and not vertical.
Site 2 - Goliaths

At the second test site a significant amount of data was lost and results are limited in this area. Nevertheless some repairs were made during the monitoring period that allowed enough results to be obtained to provide a measure of roadway behaviour. Readings from the sonic extensometer were not affected as it uses a different monitoring device. Figure 12 shows the sonic extensometer data for the Goliath pattern and Figure 13 shows the corresponding shear strain distributions. The results show a bias to the left as was the case for Site 1. However the magnitude of the displacements and the degree of shear within the overlying roof strata was increased, with maximum roof displacements increased by approximately 15 mm from the Superstrand pattern. It should also be noted that the bolt tensioner was not working correctly when the Goliaths were installed (the wedge was observed to tighten after the 1 m cut) which may have contributed to increased convergence levels in the lower part of the roof. Unfortunately limited data was available for the strain gauged bolts, but sufficient load cell data was available to assess the effect on the Goliath cables and is summarised in Table 4.

![Figure 12: Roof displacements for Goliath pattern](image1)

![Figure 13: Shear strain distribution for Goliath pattern](image2)

<table>
<thead>
<tr>
<th>Face Position</th>
<th>LHS (tonnes)</th>
<th>RHS (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Install (92m)</td>
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<tr>
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<tr>
<td>94m</td>
<td>17.0</td>
<td>9.0</td>
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<tr>
<td>96m</td>
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<td>11.0</td>
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<tr>
<td>98m</td>
<td>19.0</td>
<td>11.0</td>
</tr>
<tr>
<td>125m</td>
<td>24.0</td>
<td>17.0</td>
</tr>
</tbody>
</table>

Again a bias toward the left is present which could be observed underground. The most distinct feature however was a noticeable bulging of the strata between the rows of Goliaths at 2 m spacing. An
increase in roof deformation could therefore be observed between the cables, which are reflected in the sonic extensometer data. The row spacing of the cables therefore has an influence on roof deformation and roof stiffness. It also appears that more shear planes could develop within the cable horizon as a result of the wider spacing. Interestingly this does not appear to have created additional load on the tendons.

The Goliath pattern showed moderate levels of loading generated within the cables but with a higher degree of roof deformation when compared to the Superstrands. Closer comparison of the shear and extensometer data suggests the development of several shear planes through the cabled horizon. The results suggest that a significant proportion of the measured vertical displacement would be from horizontal movements on these shear planes, it is estimated in the range of 25% to 50%. However despite the difference in roof behaviour between the two support patterns, the overall level of damage and support loading remained within acceptable levels for the Goliath pattern.

Planning and Design Implications

A marked increase in roof deformation was measured where the Goliath cables were installed when compared to the Superstrands. In contrast the Superstrands showed higher measured loads and appeared closer to failure. The load was therefore distributed differently through the roof with each pattern. Both patterns have similar installed support density and the trial areas are only about 25 m apart. Hence the wider spacing of the Goliaths appears to have had an impact on roof deformation and associated support loading. Interestingly this particular case shows that the difference in roof behaviour would not have been reflected in an assessment based on support density and rock mass quality alone; and highlights the important role of TARPs in strata management.

The initial stability assessment suggested that there was about 10mm less to work with in the TARPs with the Goliath pattern. The monitoring program has broadly confirmed this conclusion, but the assessment has probably underestimated the degree of roof convergence in the Goliath pattern. An important observation however is the amount of horizontal movement over multiple horizons, which has contributed to the increase in measured convergence.

The propped cantilever model for the Goliath pattern does match the measured response more than the fixed beam analysis for the left side of the roadway. Only minor guttering was observed in the roadway however, suggesting that the roof beam is probably in some transient state between fixed and propped. There is also a trade-off between tendon spacing, strata damage and roof deformation as a result of a change in roadway behaviour with the different pattern. The new analysis method partly addresses the issue but not completely. Further work is required on the assessment of roof stiffness with varied tendon spacing and/or its impact on the ability to develop shear planes in the roof. The ability to assess highly bedded or micaceous roof in the GSR assessment is also under consideration.

The use of Goliaths at the wider spacing has been successful in that convergence levels, support element loading and height of softening is maintained within acceptable levels whilst the number of support elements is reduced in the development cycle. It also appears that such changes will require tailoring of the TARPs once experience with the new support is gained. This is an important outcome as the initial TARPs might need to be re-assessed based on the support pattern i.e. design and not just a change in the ground conditions. Continued optimisation appears feasible with the use of higher capacity tendons. Further monitoring will be undertaken during longwall extraction so that the impact of stress changes can be assessed.

CONCLUSIONS

An instrumented roadway trial of two sites at Grasstree Mine was undertaken to evaluate the potential for using a new high capacity tendon support at reduced density. The stability analyses and subsequent roof support trial proved successful in being able to reduce the density by increasing the spacing between cables. A measured difference in roof behaviour was however detected, with a greater degree
of shearing or horizontal movement in the roof leading to increased roof convergence for the wider pattern. The trial has provided some important insights for further development of roof support behaviour models as well as highlighting potential operational impacts on TARPs. The degree of roof movement is an important parameter in the management of roof stability and this work has provided new information on the interplay between support practice, roof conditions and stress. In this particular case, the change in roof behaviour was not sensitive to the installed support density but on other parameters such as bolt placement and spacing. It would appear that further optimisation of roof support will require assessments that consider these parameters. This is the subject of further research.

ACKNOWLEDGEMENTS

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THE EVOLUTION OF PRE-DRIVEN RECOVERY ROADWAYS AT CRINUM MINE

Yass-Marie Rutty¹, Dan Payne² and Adam Mackenzie³

ABSTRACT: Historically, the Crinum mine has experienced significant falls of ground when longwall production slowed down in preparation for recovery in weak roof areas. These conditions continue through the recovery process and result in both safety concerns and delays. When mine plans and exploration revealed that most of the future longwall recoveries were located in weak roof areas, a decision was made to try pre driven recovery roads as a solution to the problem. After completing eight pre-driven recovery roads with varying degrees of success and numerous lessons, Crinum North Mine now utilises a modified Pre-driven Recovery Roadway (PDRR) to improve the longwall take off process in weak roof areas. During mine development a standard roadway is driven where the final recovery location of each longwall is planned. After the installation of secondary support, the PDRR is backfilled with a cement-flyash mix to provide support to the roof, and confinement to the ribs and floor of the roadway. The method has been refined over the last four years to provide greater strata stability and improved operational and safety performance compared with conventional takeoffs at Crinum, and has resulted in a site record of longwall relocation in 11.5 days (pull mesh to picks in coal). This paper describes the evolution of the PDRRs from Crinum East to Crinum North including lessons from initial attempts and changes to; the secondary support regime, the operational approach during the final stages of retreat, the backfill strategy and also describes plans for the future.

INTRODUCTION

The Crinum Mine is the underground component of BHP Billiton Mitsubishi Alliance’s (BMA) Gregory Crinum Mine located north-east of Emerald, Queensland (Figure 1).

Figure 1: Gregory Crinum Location

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The Crinum mine has a history of roof control problems coming into and during longwall recoveries. Weak roof, combined with the slow mining, which is a consequence of preparing for equipment recovery, pulling mesh (which also creates tip to face issues), and the lengthy bolt up process, often results in loss of immediate roof and subsequent roof falls. In fact, approximately 50% of the first 13 longwall recoveries experienced roof fall delays, the longest of which took four months to mine the last pillar and recover the longwall. This recurring problem prompted the mine to investigate options to reduce or eliminate the problem.

The most obvious option, which had been used with some success in the US, but only 11 times in Australia with very little success (including one on which the Crinum Longwall Superintendent experienced the terrible conditions), was pre driven and supported recovery roads. These roadways are driven where the longwall will stop for recovery, reinforced with secondary support, and then mined into. After assessing the cost to drive, the cost to pre-support, and predicting the production/financial benefit of the reduced bolt up time in the recovery, it was assessed as being cost neutral (compared with a traditional recovery with no roof fall delays) and still had the risk of premature fender and roof failure being just as high as a traditional recovery (given recent experience in Australia). The mine plan was also reviewed at the time and it was decided:

1. Development could not come back and drive LW 14 pre-driven due to ventilation and conveyor locations
2. Development had not advanced enough to allow the additional driveage required
3. Longwall 15 could be recovered two pillars early to avoid a weak roof area, and longwalls 16-19 in Crinum East would be in good roof.

At this time a conscious decision was made to not employ pre-driven recovery roads at Crinum.

Two Fletcher panline bolters were purchased to minimise exposure of longwall operators to falling ground when bolting up during longwall recoveries and chutes were driven to deploy these bolters in longwalls 16-19.

However, the long term planners recognised that weak roof returned to longwall recovery locations for longwalls 20 and 21 in Crinum East and would be present for every longwall recovery in Crinum North, so a solution would still be required. Mine plans and schedules were developed to include pre-driven recovery roads for the remaining longwalls 20-28.

This paper will show that pre-driven recovery roadways can be used with success when proper care is given to their design and execution, and continuous improvement is sought, as has been the case at Crinum Mine.

**THE CRINUM PDRR STRATEGY – SUPPORT AND RECOVERY**

**Crinum East**

**PDRR20**

By the time the decision of whether to deploy a Pre Driven Recovery Road (PDRR) for Longwall 20 came around several parameters had changed:

- More development float was available and the decision was made when development was initially mining through the area
- Several more PDRRs had been employed and were successful in Australia
- The understanding of what support was required had improved
- All future longwalls were in weak roof

Longwall 20 PDRR was driven first pass (Figure 2), full seam leaving 200mm coal in the floor and taking a range of roof stone (varying between 200mm and 1m). Second pass (on the outbye side) widened the
roadway to 7.5m. A PDRR experienced consultant was commissioned to design the support, monitoring and Trigger Action Response Plans (TARPs), take part in risk assessments, and train crews. A project was run to install the support.

![Figure 2: Crinum East Mine Plan, Longwall 20 and 21 PDRRs](image)

**Longwall 20 PDRR Support and Monitoring Design**

Due to the variation in the amount of roof stone cut during development, and the concern of the longwall roof horizon upon entry, a decision was made to install a false roof in the first pass (inbye portion) of the PDRR. This false roof was made up of prefab concrete plates lifted and hung from roof bolts at the top of the coal seam horizon and then sealed and filled above with grout.

Primary roof support consisted of 8 x 2.1m long X-grade bolts per 1m during the first pass and 4 x 2.1 m long X-grade bolts per 1m during widening. Primary rib support consisted of 1 x 1.2 m and 1 x 2.1 m long X-grade bolts installed on the outbye rib and 3 x 1.2 m long fibreglass bolts per 1m on the inbye rib. The secondary support installed was; 3 x 7 m long Megabolts every 1m, 3 x 3 m long fibreglass dowels every 1 m on the inbye rib, 1 x 2.1 m long X-grade roof bolt and 1 x 1.2 m long X-grade roof bolts every 1 m on the outbye rib. As shown in Figure 3 standing support was made up of double rows of fibrecrete block cribs (with single rows for 30 m at the protected gate ends). To ensure a good flat base on which to install the fibrecrete blocks, 200 mm thick concrete plinths were constructed on the floor for every fibrecrete crib. This resulted in 1.4 MPa per metre of roadway (excluding the protected gate ends) which exceeded the required support as per the PDRR database (≥1.2 MPa).

Instrumentation consisted of Gel extensometers and roof to floor and rib to rib convergence monitoring using rotary potentiometers every 20 m cabled back to a junction box in the maingate chute road. Hydraulic stress cells installed at various depths into the fender and barrier pillar with gauges located in the barrier pillar along the maingate chute road rib. The instrumentation layout is shown in Figure 4. Geotechnical engineers were put on shift to monitor instrumentation during holing. These units were installed to monitor strata behaviour leading up to and during the final stages of longwall retreat.

**Longwall PDRR 20 – Outcome and Lessons**

Unfortunately, despite all the work done to give the Longwall 20 PDRR the best chance for success one factor was not properly accounted for. Standing support was designed to the required 1.4 MPa, however it was done using fibrecrete cribs with a capacity of 15 MPa and then built on a floor at Crinum which was approximately 4 MPa.
Mesh was pulled on the face when the fender was 8m thick with no issues. However, just prior to holing, when the fender was 2.5 m thick (Figure 5), the concrete plinths fractured at the edge of the base of the fibrecrete cribs and the cribs punched into the 4 MPa floor (Figure 6). A large crack opened up in the roof outbye the fibrecrete cribs. The longwall was stopped, and timber cribs were installed in the PDRR under the crack in the roof. Due to the damage to the floor (and concern of trying to bring the shield pontoons into the severely damaged floor), as well as the general roof instability, it was decided not to
try to remove any more of the fender until the PDRR could be backfilled with cement. Boreholes were drilled from the surface and the PDRR was fully backfilled. Three to four shears were taken to get enough space to pull shields (revealing the amount of floor punch (~1 m) and floor damage) (Figure 7) and the longwall was recovered.

At this time a few major decisions were made;

1. As Longwall 21 PDRR was already driven it was decided to learn from Longwall 20 and try again
2. It was decided not to widen Longwall 21 PDRR but to leave it at 5m wide
3. It was decided not to use fibrecrete cribs but to fully backfill the roadway.

Figure 5: Longwall 20 fender 2.5 m thick

Figure 6: Fibrecrete cribs starting to punch into weak floor
Due to the failure of PDRR 20, back analysis of the roadway was conducted by a second engineering company using FLAC 3D which modelled the failure observed. Additionally, a third consultant undertook 2D and 3D modelling of a scenario where backfill was included. They found that the fill may carry up to 50% of the induced stress as it transfers outbye, it could be assumed that a 50% reduction in floor heave would occur in LW21 when compared to LW20, all other things being equal. Their modelling suggested that PDRR21 was feasible if backfilled and would improve mining conditions.

Taking these findings into consideration, the roadway was left at 5m wide and backfilled with a flyash-cement mix of approximately 7MPa strength via a surface to seam borehole delivery system. During development of the roadway 8 x 2.1 m long X-grade bolts were installed every metre. The secondary roof support installed comprised of 1 x 6 m fully grouted Megabolt per metre near the inbye ribline and two rows of Megabolts angled over the pillar every 1.5 m on the outbye rib side. Secondary rib support was installed on the inbye rib only; 3m and 6m fibreglass dowels were installed and angled up into and above the fender at 1 m spacings. Roof mesh was also suspended from the roof at 100-200 mm below the development roof horizon (which was similar to PDRR20) to create a false roof and protect the tails of the installed secondary support (Figure 8).
The monitoring regime implemented for PDRR 21 had three main components (Figure 9):

- 5 hydraulic stress cells installed in the fender at depths of 2-10 m
- 3 hydraulic stress cells installed in the barrier pillar at depth of 5, 10 and 15 m
- 7 load cells and 3 hydraulic stress cells located within the backfill material.

Longwall 21 PDRR outcome and lessons

In line with the modelling outcomes, by maintaining a 5 m roadway width and backfilling the roadway, conditions were significantly improved during the longwall 21 recovery process from those experienced
at PDRR20. The backfill provided sufficient confinement to the fender and PDR roadway during the final stages of retreat. Operationally, the backfill presented a number of hazards:

- It was difficult to achieve satisfactory horizon control due to the seam dip
- The backfill material was sharp and angular when it failed
- When the backfill material mixed with water during recovery, trafficability was compromised.

It was also decided that hanging roof mesh to create a reinforced false roof did not achieve the intended outcome due to horizon control issues and would not be continued.

Grout was pumped into the relatively flat PDRR21 via one surface borehole and a poly pipe delivery line. This made achieving grout contact with the PDRR roof difficult and labour intensive. Several grout-to-roof voids were discovered during the longwall breakthrough. It was estimated the grout achieved approximately 90% roof contact. Multiple grout delivery boreholes were agreed for PDRR22.

The stress cells located in the fender and adjacent pillar indicated that the abutment load picked up when the face was 15 to 20 m from the cell, and the softened zone was seen to be 6 to 8 m in front of the face. The chute road took weight from 3 to 5 m outbye. A high angle shear along the outbye rib near the chutes caused the Breakerline Supports (BLS) to become iron bound during shield recovery. None of the load and stress cells located in the backfill recorded any measureable stress increase. Based on this data it was decided that vibrating wire type instrumentation would be used where possible (the hydraulic load cells would no longer be used exclusively) and strain gauges would be introduced.

Although the support design and backfill characteristics were still being refined, four days were saved during the bolt up cycle and ten days were saved on recovery time – this justified the decision to backfill future pre-driven roadways.

**Crinum North**

Crinum North had two key differences from the previous two underground mining areas (Figure 10); the orientation of longwalls, and the longwall width of 304 m (increased from 270m). The coal seam was also thicker on average which allowed 0.5-1 m of coal to be left in the floor to provide protection from the soft floor that existed at Crinum South and East.

By the time longwall mining began in the Crinum North domain, the fundamental design for the pre-driven recovery road method at Crinum had already been established. With each additional PDRR utilised for longwall recovery, new lessons were realised prompting slight refinement to the strategy for each subsequent recovery roadway.

![Figure 10: Crinum North roof uniaxial compressive strength (MPa)](image-url)
PDRR22

Longwall 22 PDRR support and monitoring design strategy

Given the success with PDRR 21, the same strategy was employed for PDRR 22. The roadway was 5m wide and 3.4 m high with the roof cut to 200-300 mm above the top of coal horizon. This would allow the longwall to retreat into the recovery road at their standard horizon without compromising the installed support (cable bolt tail lengths required to be ≤300 mm) and became the standard cut height for the remaining PDRRs at Crinum North. No hanging mesh or false roof was planned; the grout was anticipated to fall out after each cut, or be supported by additional bolts as required. The cement content in the backfill was reduced from 12% to 10% from PDRR 21 to PDRR 22 to produce a material with an Unconfined Compressive Strength (UCS) closer to that of the surrounding coal, and to allow the grout to be cut more easily and reduce the slabbing effects seen in PDRR21.

A standard eight bolt pattern of 2.1 m bolts in the roof and 1.2 m rib bolts were installed during development of PDRR 22. The secondary support regime consisted of 1 x 8 m cable bolt every 2m (or every 1 m in weaker ground) near the inbye rib, angled over the fender and two offset rows of 8m cables at 2 m spacing, both angled over the pillar. 6 m S-grade rebar bolts were installed at a shallow angle above horizontal, across and over the fender (Figure 11). 8m cable bolts were also installed in the chute roads at a density of 3 x 1 m for the first 10 m then 2 x 2 for another 20 m outbye. These cables (and a number of tin cans) were added to the plan as a result of the shearing experienced along the span of the chute roads during the removal of shields in Longwall 21.

Two vibrating wire stress cells, five hydraulic stress cells with vibrating wire transducers and three concrete embedment strain gauges were utilised to monitor stress and strain changes during longwall retreat. These cells were located around the Tailgate chute road and PDRR intersection. Additionally, four hydraulic cells were installed into the coal fender at 5 m, 2 m, 2 m and 0.3m, and one was installed at 8m into the coal pillar (Figure 12).

Figure 11: Cable bolt and rebar location, PDRR 22

![Cross Sectional View Looking from MG to TG](image)

![Plan View](image)
Longwall 22 PDRR outcome and lessons

A major roof fall occurred 9 m outbye PDRR 22 when the face was left open for a clean-up run prior to pinning the Huesker mesh. It was recognised that leaving the face open with an increased tip to face distance was not possible without additional support being installed due to the weak roof conditions (was the original reason for PDRRs and again validated the use of PDRRs). As a result, a bolt up cycle prior to pulling and pinning of the Huesker mesh was implemented for the remaining PDRRs. Cavity fill and Polyurethane (PUR) was required leading up to the PDRR. Once the affected area was consolidated, the presupported stability of the PDRR allowed the longwall to retreat safely into PDRR22 without further roof control problems which would have undoubtedly occurred given previous experience at the mine.

Stress redistribution around the PDRR was as expected within the coal fender (although greater stress changes were anticipated). The rise in stress followed by a drop (yield) immediately before the longwall hit the cell was as expected. Noticeable stress increases were shown when the wall stopped for three
days at 42 m from PDRR22 and when the initial fall occurred. Stress changes were also evident on resuming retreat after a period of the longwall standing. It was recognised that having a data logger or continuous monitoring to the surface would have enabled a more effective analysis of stress changes. PDRR22 achieved a 'tight' grout to roof fill via pumping from surface through 10 boreholes to allow for gradient and high points in the roof. This filling technique was adopted for all future PDRRs at Crinum.

Although a number of days were lost due to the fall recovery inbye PDRR 22, Figure 13 shows that the overall longwall move time was reduced, with the longwall shield recovery shortened by nine days.

**PDRR23 Overview**

The primary and secondary support remained largely the same as for PDRR 22. The key changes to the strategy were:

- The addition of a bolting cycle (in sections) prior to pulling and pinning the Huesker mesh
- The length of spiles over the fender was increased to 10 m (strands introduced to replace rebar)
- Application of a material to the roof and rib to allow the backfill material to detach from the strata; in situ trials of Tekflex and black plastic material were applied in separate areas of PDRR 23
- Due to flyash shortages from the previous supplier, a different flyash was used for PDRR23. This resulted in a 10MPa grout at LW breakthrough
- Grout was pumped up to 1km from the batch plant to PDRR 23. This resulted in valuable grout flowability and water content/grout strength lessons for future PDRRs.

The addition of the bolting cycle prior to pinning mesh was successful, reducing the previous time taken to complete "mesh to break chain" from 7 (LW21) and 12 (LW22) to just 4.5 days. The Tekflex was not successful as it did not allow the grout to fall away from the roof (Figure 14 and 15). The black plastic minimised the amount of grout remaining on the roof but prevented the longwall operators from choosing appropriate hole locations when installing 1.2 m bolts (to pin the Heusker mesh) within the PDRR. Failure of the outbye rib occurred around the maingate chute road during longwall shield recovery. The 1.2 m bolts installed during development did not provide adequate support due to the loading that occurred during the shield removal process, especially in the vicinity of the chute roads. Some 10 m spiles appeared in the face and presented an additional hazard by wrapping around the shearer drums. Inclinometers were provided to the spiling crews to improve the angle of the spiles at the collar of the drillholes.

![Figure 14: Cutting into PDRR22](image1)

![Figure 15: Cutting into PDRR23](image2)
PDRR24 Overview

The key changes from PDRR 23 to PDRR 24 were:

- 6m point anchored strands were installed by the continuous miner during development (2 x 2m vertically using the inner rigs). This allowed the second outbye row of cables to be omitted
- A combination of 2.1 m bolts and 4 m post-grouted strands (between chute roads) were installed on the outbye rib to prevent failure during shield retrieval
- Clear plastic was attached to the roof to act as a barrier between the backfill material and the roof

The longwall 24 take off process and PDRR were generally successful, though some of the same issues encountered in PDRR23 were not resolved. There was another rib fall outbye of where the 4 m cables stopped – this required cavity fill and PUR to be pumped prior to recommencing shield recovery. A number of 10 m spiles were either not installed at the correct horizon and presented in the face or bent down onto the face where blocks of roof were not supported ahead of the face. The clear plastic was not as effective due to grout from the backfill delivery boreholes breaching the roof-plastic interface.

BACKFILL STRATEGY AND RESULTS

Prior to grout filling PDRR21 substantial time and money was invested in differing grout mixes to achieve the required strength and flowability specifications. Grout mix considerations included:

- Flyash supplier – multiple flyash sources were trialled
- Grout composition – differing cement/flyash content
- Water content – achieve required grout flowability
- Water quality – salts and Total Dissolved Solids (TDS) will impact grout strength
- Curing times – cost benefits achieved by reducing cement content and allowing longer curing time.

The original trial data combined with the developed strategy after multiple PDRRs has resulted in the following decisions being made:

1. The flyash now used at Crinum (and since adopted at Broadmeadow Mine) consists of a light fine grained ash of consistent size and composition. This provides grout strength predictability, ease of pumping and consistent water requirements
2. The original 7 MPa grout specification has increased to 10-14 MPa. While a harder grout has the potential for brittle/sharp edges during cutting it generally shears away from the roof better and has improved qualities through the coal processing plant i.e. less daughter particle creation
3. Water content can be altered and tested to achieve the required grout pumping distance (from plant to PDRR) without negatively impacting the grout strength
4. Water required to produce grout with predictable strengths must have known salts and TDS levels. Water containing high salts or TDS will make grout strength prediction difficult
5. By scheduling longer curing times prior to longwall breakthrough, cement contents can be reduced. This results in a cheaper grout that still achieves the required strength
6. Grout delivery via multiple boreholes is a quicker, less labour intensive and more cost effective filling technique that provides a tight filled PDRR (>95% filled). Subject to borehole depth and casing, the cost of 10 grout delivery boreholes is less than one borehole with an underground network of delivery pipes and pipe install/ fill supervision labour
7. Consideration and some trials using other ‘filler’ materials other than flyash have been undertaken to further reduce PDRR costs e.g. fine coal tailings, aerated grouts.
PDRR 27 – THE FINAL ITERATION AT CRINUM MINE

All pre-driven recovery roadways for the life of mine at Crinum have been developed, supported and backfilled. The final PDRR to be prepared at Crinum is longwall 27. Though not yet mined, this roadway represents the final iteration of the method to be applied at the mine.

The 10 m JSS cables over the fender were replaced with 9 m length self-drilling bolts made up of 5 x 1.5 m hollow steel bolts coupled to 1 x 1.5 m hollow fibreglass bolts at the collar of the hole. The ability to couple of self-drilling bolt components not just with like materials (steel coupled with steel) but also steel and fibreglass has enabled the mine to remediate the issue of variable longwall horizon immediately prior to breaching the roadway. Another advantage of this system is that due to the much stiffer nature of the self-drilling bolts as opposed to the strands, they are not anticipated to present the same hazard experienced in previous pre-driven roadways where the ductile strands have wrapped around the shearer drums and provided little or no reinforcement to the roof once partially exposed. Advantages were also seen in the ease and quality of installation of these bolts.

The approach during the final stages of longwall retreat will remain largely the same utilising a pre-bolt up cycle (two rows 2.1 m bolts) and three rows of 1.2 m bolts to pin the Huesker mesh within the PDRR. The Huesker mesh will continue to provide protection during shield recovery by catching any small slabs of backfill material that remain on the roof after the final shear has been taken. Longer lengths of Huesker mesh may also be ordered so it can continue down the face and be pinned in place using bolts and/or straps where backfill material is unable to be removed from the final ribline.

CONCLUSIONS AND RECOMMENDATIONS

Significant progress has been made at the Crinum mine in the application of pre-driven recovery roads as evidenced by the evolution of the support regime, approach strategy, and backfill composition, as well as improved safety and recovery time. There are still a number of improvements to future PDRR design and implementation that should be considered.

1. Silent seal, or the like, the face rib so that the grout comes away from it and doesn’t hold on to the mesh and bolts temporarily, and subsequently fall away during shield recovery
2. Install a trial of standing support like fibrecrete blocks or pumpable cribs and backfill the roadway to ½-3/4 height. This allows the floor bearing capacity required, the roof support required, the rib confinement and fender support required, the floor heave control required, and then delivers a clean bolted roof as the backfill doesn’t hang on to roof bolts and mesh and fall away later. Crinum was planning to do a 50 m trial section to prove the concept but ran out of longwalls
3. A full width PDRR (14 m) to eliminate the requirement to put up additional bolts
4. Evaluate the results of utilising a combination of steel and cuttable support above and within the fender
5. Continuous improvement of the backfill strategy – consideration and trials using ‘filler’ materials other than flyash to reduce cost, refinement of strength based on the strength of in situ strata.

The success of pre-driven recovery roadways at BMA’s Crinum mine as compared to conventional longwall recovery within a weak roof environment has shown that when proper consideration is given to their design and execution, and continuous improvement is sought, PDRRs are a worthwhile and necessary strategy to ensure a safe and effective longwall take off.
OPTIMUM SIZE OF COAL PILLAR DIMENSIONS SERVING MECHANISED CAVING LONGWALL FACE IN A THICK SEAM

Guanyu Yang, Cun Zhang, Runlong Yan and Shuai Jia

Abstract: Chain Pillars serving longwall face is an important factor influencing the stability of the roadway surrounding rock in the fully mechanised caving longwall face. The reasonable width of the section coal pillar is proposed, combined with the theoretical analysis, numerical simulation and field measurement for the fully mechanised caving longwall face in thick seam in Jinzhuang coal mine. Results of the numerical simulation show that when the width of the section pillar increases to 24 m, there is an 8 m wide elastic zone in the coal pillar and a saddle-shaped vertical stress distribution, which shows that the coal pillar can maintain its stability. So the reasonable width of the coal pillar is 24 m, which is close to the result of theoretical calculation (23 m). The field observations also illustrate that the 30 m wide coal pillars in the first mining face is too large resulting in a waste of resources.

INTRODUCTION

Chain Pillars serving longwall face is an important factor influencing the stability of a roadway serving the fully -mechanised longwall face caving. Research has been carried out for the design of optimum coal pillar width. The coal pillar design theory and equation proposed by Wilson(1980) is the most widely applied theory, Wang et al., (2002) and Xu et al., (2005) have introduced creep and constitutive relationships and analysed the long-term deformation and stability of the coal pillar, thus establishing the necessary conditions for maintaining coal pillar stability; Bai et al., (2004), using numerical calculations, studied the relationship between the stability of the narrow coal pillar, the width of coal pillar, the mechanical property of coal and rock mass, and provided the indexes to evaluate the stability of the coal pillar from the perspective of materials and structures; Zheng et al., (2012) studied the stress fields distribution rules in the mining process of chain pillars of different widths along the goaf-side entry drive and proposed two influence factors; disturbance influence of roadway driving and advanced mining influence of working face. These two factors were suggested to be considered when deciding the optimum coal pillar width along a goaf-side road entry drive.

Although there are various designs of optimum width of coal pillars, nevertheless they are all limited to the design of coal pillar in the longwall face for certain conditions and there is seldom any field measurement being carried out to verify them.

Based on the engineering background of working faces 8203 and 8204 as the first mining face with super high seams in Jinzhuang coal mine of the Datong mining areas, this paper reports on research methods of combining theoretical analysis, field measurements and numerous simulations, which were used to determine the optimum width of a coal pillar in a fully mechanised first longwall caving face in super high seams and verified its rationalisation by field measurement.

GEOLOGICAL ASPECTS

The burial depth of the Jinzhuang coal mined seam is 290~340 m, and the dip angle of the coal seam is 3~4°. The working seam is coal seam 3-5# with stable coal layers. The lithological character is shown in Table 1.
Table 1: The Lithological Characters

<table>
<thead>
<tr>
<th>Sequence Number</th>
<th>Position</th>
<th>Thickness/m</th>
<th>Lithology</th>
<th>Lithological Characters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Overlying rock strata</td>
<td>20</td>
<td>Medium grained sandstone</td>
<td>Layard, medium grained texture, massive structure</td>
</tr>
<tr>
<td>2</td>
<td>Upper roof</td>
<td>12</td>
<td>Gritstone containing gravels</td>
<td>Layard, coarse grained texture, massive structure</td>
</tr>
<tr>
<td>3</td>
<td>Immediate roof</td>
<td>0.8</td>
<td>Fine sandstone</td>
<td>Layard—grey, fine grained texture, massive structure</td>
</tr>
<tr>
<td>4</td>
<td>False roof</td>
<td>0.5</td>
<td>Mudstone</td>
<td>ash black, bedding joint</td>
</tr>
<tr>
<td>5</td>
<td>Coal 3#</td>
<td>9</td>
<td>Coal</td>
<td>black, massive structure, pitchy luster, and brownish black striation</td>
</tr>
<tr>
<td>6</td>
<td>Dirt band</td>
<td>0.6</td>
<td>Mudstone</td>
<td>ash black, containing a large amount of plant fossil fragment</td>
</tr>
<tr>
<td>7</td>
<td>Coal 5#</td>
<td>7</td>
<td>Coal</td>
<td>black, semimonocoque, layer texture, massive structure</td>
</tr>
<tr>
<td>8</td>
<td>Director floor</td>
<td>5</td>
<td>Carbone mudstone</td>
<td>black, pelitic texture, massive structure</td>
</tr>
<tr>
<td>9</td>
<td>Base floor</td>
<td>15</td>
<td>Silty mudstone</td>
<td>grey—ash black, aleuritic texture—pelitic texture, massive structure, diagonal structure development</td>
</tr>
</tbody>
</table>

The longwall top coal caving method was used in working faces 8203 and 8204. The mining thickness was 3.9 m and the top coal average thickness was 11.55 m. The ratio of mining height to caving height was 1:2.96. The working face 8203 was mined firstly, followed by the working face 8204. There was 30 m section pillar which ensured the roadway kept stable during mining. The detail is shown in Figure 1.

The main entry roadway 2203 and main return air roadway 5204 were arranged along the floor of the coal seam of 3 to 5, and a second return air roadway 5204-1 roof was excavated along the stable strata of coal seam 3-5# roof.

![Figure 1: Plane figure of the working face layout](image)

THEORETICAL ANALYSIS OF REASONABLE WIDTH OF SECTION COAL PILLAR

The front abutment support pressure in front of the coal face can be affected by the mining activity and has a great influence on the deformation of the surrounding roadway rock layers. Therefore, the determination of optimum width of the coal pillar would ensure the stability of the coal pillar and roadway.
surrounding rock formation when the working face advances. An optimum coal pillar width left between two roadways is left to ensure the stability of roadways in the service period so as to achieve the regular production of the working face.

Based on the most widely applied coal pillar design theory (Wilson, 1980), researchers in China examined the calculation equation for designing the section coal pillar. The ultimate strength that the coal pillar can bear is:

$$\sigma_u = \frac{2C \cos \varphi}{1 - \sin \varphi} + \frac{1 + \sin \varphi}{1 - \sin \varphi} (\lambda \varphi H \gamma (a - 0.00492kMH))L$$

The actual ultimate strength that the coal pillar has to withstand is:

$$\sigma_a = \gamma H (a + \frac{b}{2} - \frac{b}{0.6H})L$$

Where; $a$ is the width of coal pillar (m); $b$ is the width of working face (m); $H$ is depth of mining (m); $\gamma$ is the average volume force of underlying strata (kN/m$^3$); $C$ is the cohesion of coal body (MPa); $\varphi$ is the internal friction angle (°); $\lambda$ is the factor of stress concentration which is 0.4~0.8 and based on experiments; $K$ is 0.225~0.25; $M$ is the mining thickness (m); $L$ is the length of section coal pillar (m).

The necessary condition of keeping coal pillar stable is:

$$\sigma_p \leq \sigma_u$$

According to the above equation, the width of coal pillar should satisfy conditions in the following:

$$a \geq \frac{\gamma H b}{2} \left(2 - \frac{b}{0.6H}\right) + 0.00492kMH(\frac{2C \cos \varphi}{1 - \sin \varphi} + \frac{1 + \sin \varphi}{1 - \sin \varphi} \lambda \varphi H - \gamma H)$$

According to the practical data of working face 8203: $b=220$ m, $\varphi=35^\circ$, $M=16$ m, $K=0.225$, $\lambda=0.8$, $\gamma=19$ kN/m$^3$, $H=300$ m, $C=3.07$ MPa, inserted into equation 4, the width of coal pillar can be calculated at $a \geq 22.9$ m, say $\geq 23$ m. Thus for the stated conditions the actual ultimate strength that the coal pillar withstand will not be over its ultimate strength, thus the coal pillar maintains stability.

**THE NUMERICAL CALCULATION OF THE REASONABLE WIDTH OF SECTION COAL PILLAR**

The strata of roof and floor in this simulation project adopts the Mohr-Coulomb model, the coal seam adopts a strain-softening model and the goaf adopts an complete elastic model.

Software FLAC3D was used to analyse the evolution rules of coal pillar stress and plastic zone development of the working face on two sides in different mines. In the simulation, the width of the coal pillar has adopted 16, 20, 24 and 30 m, left and right working faces are excavated to 20, 50, 80, 90, 100, 110, 120, 150, 180 and 200 m, the evolution rules of coal pillar stress and plastic zone in 100 m section were analysed. The lengths of the two coal pillar of the working faces were taken the same. Details are shown in Figure 2.

Figure 3 shows changes of plastic stress in the coal pillar. Only the corresponding section of coal pillar is shown and the width of each grid is 1 m. The coal pillar was analysed according to the evolution of plastic zone and stress in coal pillars with the advancing coal face of length 100 m. From Figures 3 to 5, the development of plastic zone is in the left side of in the coal pillar is and the right side is its corresponding stress evolution.
The changes of plastic stress in 16 m and 20 m wide coal pillar

When the width of the coal pillar increases from 16 m to 20 m, the bearing capacity will increase, causing pillar failure during the mining process. From the perspective of stress, the peak stress value on 20 m wide, coal pillar will be relatively smaller than that of 16 m coal pillar width. Note that the overall residual strength of 20 m failed coal pillar is also larger than that of 16 m coal pillar.

The changes of plastic stress in 24 m wide coal pillar

As is shown in Figure 3 and Figure 4, when the width of coal pillar increases from 20m to 24m, the development of plastic zone and the stress in coal pillar are quite different from the previous situation. In the advancing working face, the plastic zone of coal pillar, that is continuously being developed, does not spread all over the coal pillar. In terms of stress, the stress peak value will increase gradually and continuously move inwards, however, the left and right peak points do not overlap. The result clearly indicates that there is an elastic zone with small stress between the two peak point, so the final distribution of stress in the coal pillar is saddle-shaped with the two sides of the coal pillar will been destroyed as the stress in the middle part of the coal pillar increase. This, however, will be an elastic zone with the coal pillar maintains its stability.

The changes of plastic stress in coal pillar of 30 m in width
As is shown in Figure 5, when the working face finishes there remains a 17 m wide elastic zone in the coal pillar core and the stress distribution is roughly "saddle-shaped". Therefore, it is clear that the 30 m wide coal pillar is better, which seldom loses stability.

In conclusion, as the width of coal pillar increases from 24 m to 30 m, the stress in the coal pillar does not change significantly, which demonstrates that further increase in coal pillar width >24 m, the increase of coal pillar width has less effect on the bearing capacity of the coal pillar, therefore, 24 m wide coal pillar can meet the project requirements. Leaving large pillars will lead to sterilisation of the mineable coal which is an uneconomic endeavour.

FIELD MEASUREMENTS OF THE STRESS AND DESTRUCTION OF COAL PILLAR

Borehole stressmeters, particularly borehole hydraulic pressure cells were used to monitor coal pillar stress and the advancing support pressure of the working face. The coal pillar width between longwall faces 8203 and 8204 was 30 m, the length of working face was 220 m and the longwall panel advance length was 1450 m. Borehole stressmeters were installed at the locations commencing 800 m away from the longwall face. The stress meters were installed at 1.5 m, 3 m, 5 m, 10 m, 15 m and 5 m along the pillar length as shown in Figure 6 and at the height of 1.6m above the roadway floor.

Figure 6: The layout of borehole stress meter

The working face was actually advanced about 700 m when the borehole stressmeter installtion arrangement was completed, and with the borehole stress meter of 3 m being some100m away from the working face. With the advance of the working face, changes in stress levels in each location with respect to their positions from the working longwall face are shown in Figure 7.

Figure 7: The rule of actual measured change of each borehole stress meter
As is shown in Figure 7, after the borehole stress meters were installed, the stress begins to decline slightly; this is because of the initial stress plug oil compressibility. When the working face was 80 m away from borehole stress meter, the stress begins to increase slowly; when the working face is about 60 m away from the measured zone (the measured zone starts with the borehole of 3 m in depth), the borehole stress rises linearly which indicates the advanced support pressure of the working face has already influenced the observation area and also proves that it is reasonable for the two roadways in the working face to be supported for 50 m in advance. When the working face is about 16 m away from observation borehole, the borehole stress increases rapidly, when this distance reduces to 14 m, the increasing amount of the stress is the largest, which indicates that the approaching support pressure reached the peak value in this place. The field measurement and analysis demonstrated that the numerical simulating results were correct and a 30 m wide a coal pillar in coal mine is larger, thus sterilising more coal in the pillar.

CONCLUSIONS

The stability rules of coal pillar in an operating mining roadway are studied experimentally, theoretically as well as by numerical simulation. It was found that the minimum pillar width that maintained a stable working environment was 24 m. This minimum pillar width allowed an elastic zone in the pillar core sufficient to maintain stable working conditions. Thus, the use of 30 m wide pillars sterilises a significant amount of coal in the pillar unnecessarily.

REFERENCES

STUDY ON THE ANALYSIS METHOD OF SWELLING DEFORMATION OF PROTECTED SEAM DURING PROTECTIVE SEAM MINING

Huo Bingjie¹²³, Lu Yangbo¹, Tang Guoshui¹, Fan Zhanglei¹, Wang Zhe¹ and Zhou Kunyou¹

Abstract: In view of the study on swelling deformation analysis method of protected seam during the mining process of protective seam, the analysis method of “four invariant points around area” is put forward for the first time. The method determines the swelling deformation of protected seam and analyzes it from the perspective of plane by analyzing the variability of “four invariant points around area” of protected seam before and after the mining of protective seam. Monitoring scheme and area analysis and calculation method are respectively designed applied in coal mine and laboratory; the monitor of “four invariant points around area” has been realized in the mining practice by arranging two measuring lines in the roof and floor of protected seam. The study scheme is designed to analyze the swelling deformation of the protected seam by the application of “four invariant points around area” in the engineering practice; the theoretical calculation method of irregular “four invariant points around area” after swelling deformation of the protected seam is put forward under laboratory conditions based on the Freeman boundary encode vector and measuring the length of quadrilateral side directly with the vernier caliper.; the reasonable scale of the “four invariant points around area” is discussed, it is suggested that different “four invariant points around area” should be established with different scale of 1 times, 1/2 times, 1/4 times and 1/8 times thickness of coal seam. The study shows that the method of “four invariant points around area” of swelling deformation is more accurate than the analysis method of “two fixed-point”; the more cells are divided at 1 times of the thickness of coal seam, the higher the accuracy of calculation is.

INTRODUCTION

With the increase of the depth of coal mining, ground stress, gas pressure and temperature rises, and this has seriously affected the safe and efficient production of coal mines. Mining the protective seam can reduce the original rock stress of the overlying protected seam, release elastic potential energy, make the protective seam and the surrounding rock produce swelling deformation, develop fractures, increase the permeability coefficient, release the adsorption gas from the protected seam and surrounding rock, provide fracture channels for the gas flow and provide the conditions for gas desorption-diffusion-seepage, that is, protective seam mining has the effects of pressure-relief, increasing permeability and fluidity (Ma et al., 2012; Xu 2011; Tu et al., 2006; Yuan Liang 2009; Xie et al., 2014) and Yuan Liang, et al, 2013). Mining the protective seam is one of the important measures to solve the problem of coal and gas outburst.

The permeability of unloaded rock mass during the process of mining the protective seam mainly depends on the swelling deformation rate of the protected seam. Scholars at home and abroad have carried out a lot of researches on the swelling deformation characteristics of the protected seam, Tu Min et al., (2006, 2007) has studied and divided the deformation with similar material simulation experiment and concludes the law that swelling deformation will increase the permeability. Zhang Shujin et al., (2013) with asimilar material simulation experiment, has analyzed the swelling deformation law of the mining seam in dual protective seam of seam group and concludes that after mining dual protective

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The swelling deformation curve of protected seam is "M" type in the direction of trend. Ma Zhanguo et al., (2008) has studied the motion rule of mining-induced overburden rock and the stress and deformation rule of the protected seam during the coal seam advancement process with physical simulation experiment. Shi Biming et al., (2008), with a similar material simulation experiment, has analyzed the deformation characteristics in a vertical and horizontal direction and the influence of distance between the protective and overlying protected seam on the protective effect. Valliappan (1997) and Dziurzynski (2001) have studied and analyzed the overburden pressure relief deformation caused by the protective seam mining with numerical simulation.

The above research is mainly aimed at the characteristics of swelling deformation of protected seam and the important influence of swelling deformation on the protective effect during the process of protective seam mining, for the characteristic analysis of swelling deformation, the swelling deformation of the protected seam is represented by the two fixed distance changes in the normal direction of the protected seam. This method only considers the deformation characteristics of the protected seam in normal direction from the view of distance change between the “two fixed points”, tensile creep, in fact, has occurred in tendency and normal direction of protected coal seam during the mining process of protective seam. Due to that the traditional coal seam swelling deformation analysis method reflects the swelling deformation of the protected seam by calculating the distance change between the two measuring points in the normal direction of the protected seam before and after the mining of the protective seam mining, the result can not reflect the lateral deformation of the protected seam and the swelling deformation characteristics of coal and rock body accurately. This paper puts forward the analysis method of “four fixed point area” and study on the swelling deformation characteristics of the protected seam from the two dimensional angle, In engineering practice and similar material simulation experiment, the area of the quadrangle is determined with the four constant measuring points in the protected seam, the swelling deformation rule is analyzed with analyzing the change amount of “four invariant points around area” before and after the mining of protective seam. Therefore, the analysis method of “four invariant points around area” uses the area deformation method of the protected seam instead of the traditional method of the distance change between the two points to analyze the swelling deformation; the result is more reasonable and practical than the traditional swelling deformation analysis method.

Now, the variation of the distance between “two fixed-points” is usually used to analyze the swelling deformation of the protected seam, the characteristics of swelling deformation of pressure relief coal is achieved by analyzing the distance change of the “two fixed-points” in the normal direction, before the protective seam mined and after.

In engineering practice, the measuring boreholes are arranged in the top and bottom of the protected seam through the panel crossheading or bed plate tunnel of the protective seam, and a displacement measuring point is arranged in the top and floor of the protected seam to measurement the swelling deformation of protected seam during the process of the protective seam mining, Dziurzynski and Krach (2011) and Du (2011), as shown in Figure 1. The displacement curves about the change of normal displacement with time and working face position in the roof and floor of the protected seam was obtained by the data of recording, and the swelling deformation characteristic is determined.

In the process of experiment simulation of similar materials, the equal distance measuring points are generally arranged on the roof of the protected seam. With the mining of the protection seam, the movement and deformation of measuring points on the roof and floor of protected seam are measured in the normal direction by using the displacement meter. The difference of displacement variation of the two points along the normal direction is expressed as the swelling deformation of the protected seam Ma et al., (2008), as shown in Figure 2.
Figure 1: Analysis the swelling deformation of protected seam based on distance variation of two fixed point

ANALYSIS METHOD OF FOUR INVARIANT POINTS AROUND AREA OF SWELLING DEFORMATION

The scheme design of measurement and Calculation method of “four invariant points around area in the experiment of similar material simulation

The scheme design of measurement of “four invariant points around area" in engineering practice is that, two groups of survey lines of "two invariant point" are arranged in the top and bottom of the protected seam through the panel crossheading or bed plate tunnel of protective seam. Determine four invariant points in the protected seam, analyzing the change of "four invariant points around area", before and after the protection seam mining, and realize the measurement of “four invariant points around area” (Figure 3).

The measurement scheme and theoretical calculation of “four invariant points around area” of swelling deformation

The calculation method of "four invariant points around area" of swelling deformation in engineering practice is that the length of the sides 1l and12 along the dip and four fixed-point coordinates of “four invariant points around area” are gained by measuring the length of hole in the protected seam and the layout position of measuring borehole and then the “four invariant points around area " before swelling deformation in the protected seam can be calculated by the “irregular variability software V2.0.4” with the coordinate of each point as shown in Figure 4. Because of the low swelling deformation rate of protected seam, before the protective seam is mined and after, the approximate selection of the “four invariant points around area” is the rule. the down displacement amount (m1) at the top and the down displacement amount (m2) at the bottom of the left survey line of "two invariant points " are obtained according to measurement of the displacement amount of the roof and floor side deformation of the left hole 1#; Simultaneously, the displacement amount (m3) and (m4) of the two points on the 2# can be measured. Among them, the black area around the field is the area before the swelling deformation, the red is the area after the swelling deformation. With the low swelling deformation value of the protective seam in engineering practice, it can be approximately thought that the upper and lower boundary of the surrounding area after the swelling deformation of the pressure relief coal seam is the line between the upper two points and the down two points. The coordinates of the four fixed points after swelling...
deformation can be obtained by calculation, taking the left lower corner of the enclosed area as the origin of coordinates. The area (s) of the enclosed area after the swelling deformation of the protected seam can be calculated by irregular variability software. According to the value of the change of the “four invariant points around area”, the swelling deformation rate of the protected seam is:

$$n_i = \frac{s' - s}{s} \times 100\%$$

(1)

Figure: 3 Schematic diagram of measuring hole layout

Figure: 4 diagram of four invariant points around the area of before swelling deformation and after

The scheme design of measurement and Calculation method of “four invariant points around area in the experiment of similar material simulation

Based on the visualizing characteristics of similar material simulation, in order to monitor the swelling deformation of the protected seam during the process of mining, the displacement monitoring points are selected on the protective seam. Choosing reasonable scale on the protected seam to arrange monitoring points and pasting the non-coding mark point on the monitoring points. Select four invariant points around area as the research object and make sure the area is quadrilateral, measure the displacement in the direction of dip and normal to the coal seam and the coordinate of four fixed points during the process of swelling deformation of the protected seam by using the XJTUDP software, as shown in Figure 5. Randomly selected four points on the protected seam are $A_1, A_2, A_3$ and $A_4$ respectively, before the swelling deformation of the protected seam. Assume the four points, after swelling deformation, are $A_1', A_2', A_3'$ and $A_4'$ (Figure 6).

For the experiment of similar material simulation, a steel nail can be used to arrange the four fixed points in different scale in the protected seam and the surrounding region, record the changes of “four fixed points around area” in the process of mining the protective seam and then analyze the swelling deformation rate. Because swelling deformation rate of the protected seam is small under protection seam mining, to simplify the calculation, the calculation model of the “four fixed-point area” is considered as a quadrilateral in the calculation and the length between each two measuring points in the quadrilateral can be fetched directly by using the vernier calipers, the swelling deformation ratio is gained by calculation of the “four invariant points around area” difference before and after the swelling deformation of the protected seam.

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Figure 5: Partial enlargement of mark points

Figure: The diagram of analysis the four point surrounded area

Under the influence of disturbance stress, the four invariant points around area is a irregular quadrilateral after swelling deformation of the protected seam, With the mining of protective seam, using digital camera to take pictures for the area of the "four invariant points around area" obtained the boundary of image and coordinate values of each pixel points on the boundary through image segmentation, boundary extraction and tracking, \((i \text{ means the } i \text{ pixel point}, \ i=0\cdots N−1 )\). The starting point is set in the down-left side of target boundary based on the habit of tracking boundary algorithm, as shown in Figure 7. The vector relation of the adjacent points' position of the boundary target can be expressed by Freeman chain code, the possible eight colligated direction between two adjacent pixels on the boundary curve is defined as 0°,45°,90°,135°,180°,225°,270°,315°.The position relationship between the previous pixel point and the position as well as the position relationship between the pixel and the next pixel are respectively defined based on the vector relation making principle of Freeman chain code. After the corresponding sum of the vector and the VC program calculation, we can get the quadrilateral \(A_1A_2A_3A_4\) area:

\[
S' = \sum_{i=0}^{N-1} \text{pixl}[i] \cdot x \cdot B[i] + N_1
\]

where \(N_1\) is represented as the number of 1 in \(B[i]\), \(B[i]\) representation the vector relation making principle of Freeman chain code.
The swelling deformation rate of the protected seam is obtained by the area difference between the front and back of protective seam,

\[ n_2 = \frac{S - S'}{S} \times 100\% \]  \hspace{1cm} (4)

where \( n_2 \) is the swelling deformation rate, \( S \) is the “four fixed points around area” of the protected seam before swelling deformation, \( S' \) is the “four fixed points around area” after swelling deformation.

![Figure 7: Object's figure of boundary](image)

The reasonable scale analysis of the four invariant points around area

Because of the different thickness of the protective seam, the scale of “four invariant points around area” in engineering applications and similar material simulation experiment should be chosen reasonably to calculate the swelling deformation of the protected seam accurately. Therefore, it is initially proposed to analyze the swelling deformation ratio of the protected seam by the basic unit of four invariant points around area” with 1 times, 1/2 times, 1/4 times, 1/8 times, and other different scales of the thickness of the protected seam, as shown in Figure 8.

In engineering practice, the seams, usually thick coal seam (thickness is 3.5~7.99m) or very thick coal (thickness ≥8m), need protective seam mining technology to prevent failure. To choose the “four invariant points around area” conveniently and increase the corresponding calculation accuracy, the thick coal seam is divided into units by the thickness of 1 times, 1/2 times or 1/4 times of the coal seam thickness; the extremely thick coal seam is divided into units by the thickness of 1/2 times, 1/4 times or 1/8 times of the coal seam thickness; Among them, the “four invariant points around area” in 1 times scale of coal seam thickness is divided into 4 units by the geometric scale of 1/2 times of the coal seam thickness; simultaneously, the “four invariant points around area” in 1 times scale of coal seam thickness is divided into 16 units by the geometric scale of 1/4 times of the coal seam thickness, the “four invariant points around area” in 1 times scale of coal seam thickness is divided into 64 units by the geometric scale of 1/8 times of the coal seam thickness. The more the area units are in the 1 times scale of coal seam thickness, the higher is the accuracy of the calculation.
Consider 1.0 m of thickness of the protected seam as example, analysis of the swelling deformation, by using the analysis method of two invariant point distance variation and the analysis method of four invariant point area and undertake a comparative analyse the accuracy of two methods. The coal and rock swelling deformation ratio of analysis unit is 5%, calculated by the analysis method of two point distance variation, that is, coal and rock increases by 0.005m in the normal direction. Assuming the Poisson's ratio ($\nu$) is 0.36, then the unit length of coal body increases by 0.0018m alone the dip direction of the protected seam by using the Poisson's ratio for estimation.

Since the coal and rock swelling deformation in the protected seam is small, to simplify the calculation, the four points around domain area is assumed as regular quadrilateral. Then the four invariant points around area is 1.006809 m$^2$ after the swelling deformation. Because of the "four invariant points around area" is 1.006809 m$^2$ in the original state, so swelling deformation ratio, calculated by the analysis method of "four invariant points around area will be

$$n = \frac{1.006809 - 1}{1} \times 100\% = 6.809\%$$  \hspace{1cm} (5)

Therefore, by using the analysis method of "the four points around domain area the swelling deformation value will be reasonably accurate and reflects compared with the swelling deformation characteristics of the protected seam than the traditional "two point" distance variation analysis method.

CONCLUSION

1. The characteristic of swelling deformation of the protected seam is one of the main indexes of protection effect investigation, this paper puts forward the "four invariant points around area" method for the swelling deformation analysis, analyzes the swelling deformation characteristics of the protected seam from two-dimensional view and considers the deformation effect of protected seam in the dip and the normal direction comprehensively, the result is more accurate and practical.

2. The study scheme and area calculation method are designed to analyze the swelling deformation characteristics of the protected seam by the application of "four invariant points around area" in the engineering practice; The theoretical calculation method of irregular "four invariant points around area" after swelling deformation of the protected seam is put forward based on the Freeman boundary encode vector under laboratory conditions and the area change calculation method by measuring the length of quadrilateral side directly with the vernier caliper.

3. The reasonable scale of the "four site area" is discussed and the "four fixed-pointed area" is established with the scale of 1 times, 1/2 times, 1/4 times, 1/8 times of the thickness of coal seam. The more the area units are in the 1 times scale of coal seam thickness, the higher the accuracy of the calculation is; it is initially proposed that the area unit division should be done by the 1 times, 1/2 times or 1/4 times scale of the seam thickness if the protected seam is thick coal seam and done by 1/2 times, 1/4 times or 1/8 times of the seam thickness if it is an extremely thick coal seam.
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STUDY ON RIB SPALLING MECHANISM AND SPALLING DEPTH IN LARGE MINING HEIGHT FULLY-MECHANIZED FACE

Hong-wei Zhang1,2, Xing FU1,2 and Yu-zhi Shen1

Abstract: Due to the influence of the high intensity mining operation in the Shendong mining area located in Ordos City, Inner Mongolia Region of China, the problem of rib spalling in large mining height and fully-mechanized working face production is increasingly becoming more serious. The occurrence and location of the maximum depth of rib spalling is studied in the 12301 working face of Shangwan coal mine in the Shendong mining area taken as an example. The comprehensive methods of theoretical analysis, numerical simulation and field studies are adopted in this study. The gradual deterioration characteristics of rib spalling was studied using the ‘thin plate’ mechanical model of the working face and the empirical equation. The study showed that the rib instability was generally located below the roof at the position of 0.578 times of the mining height. The theoretical calculation showed that the maximum depth of rib spalling was 0.98 m~1.61 m at the 12301 working face, and the initial rib spalling started 2.53 m below the roof, which is basically consistent with the results of the numerical simulation and the statistical analysis of data from the field. The study provides the foundation for the future control measures of rib spall.

INTRODUCTION

In recent years, with the trend of nationalization and heavy-industrialisation of fully-mechanized mining equipment, the fully-mechanized mining technology is widely applied in Shendong, Jincheng and Datong mining areas. The technology has achieved considerable economic and social benefits due to its high resource recovery rate and large production capacity. This technology is considered to be one of the best coal mining methods for safe and highly efficient mining of the thick coal seams in China (Gao Jin and He Hai-tao 2010). However, many engineering field practices show that accidents due to rib spall are more likely to occur in the working face due to the increase of mining height. Spalling could also contribute to the increase of unsupported roof span in front of the support line causing roof falls that can seriously affect the normal advancing speed and cyclic operations of the working face. Therefore the rib spall is one of the most serious problems that must be resolved. Only if the mechanism and the depth of rib spalling are fully understood and managed then the potential of large mining fully-mechanized working face can be fully achieved (Meng Chao 2013 and Zhu Yong-jian and Feng Tao 2012).


The average depth of 1-2# coal seam is 240 m in the Shangwan coal mine, Shendong mining area. The large mining height using a fully-mechanized mining technology was applied in the working faces of the 1-2# coal seam in the 3rd panel. The large area of face spall occurred repeatedly during the mining process, which seriously affected the normal production of the working face. The “thin plate” mechanical
model and the deflection equation to predict the working face rib deformation was established by using fracture mechanics as the theoretical basis for the longwall working face LW12301. This work examined the opening and extension of coal fractures for the large mining height. Using theoretical analysis and numerical simulations, the position of rib instability and the mode of failure is researched here and the depth of rib spall and failure mode determined.

MINING CONDITION OF 12301 WORKING FACE

Shangwan coal mine is one of the main producing mines of the Shenhua Shendong Coal Mine Group Co, Ltd, which is located in the southeast of Erdos City, Inner Mongolia. There are five mineable seams (1-2 up, 1-2, 1-2 down, 2-2 and 3-1 seams). The total thickness of all seams is 16.50 m, of which the 1-2 coal seam is divided into 4 panels. The 1-2 coal seam is nearly horizontal and about 6.2 m thick. The geographical position of Shangwan coal mine is shown in Figure 1.

LW12301 is the first working face of panel 3 in 1-2 coal seam, which is operated using a fully-mechanized longwall mining method. The mining height is 6.0 m, the face length is 249 m and the mineable advancing panel is 4948 m long. The roadway arrangement of LW12301 is shown in Figure 2. The thickness of the sandy mudstone immediate roof ranges from 0.63 m to 3.87 m. The thickness of the fine-sandstone main roof is 1.3~4.2 m and is the main component of sandstone. The thickness of immediate bottom is 0.56~2.11 m with mudstone easily softened with water. The site investigations revealed that the large area of face spall exploded into the working area several times during the periodic weighting with the fish scale like face appearance. The maximum depth of rib spall was 1.7 m, which decreased the normal shearer and hydraulic support efficiency and posed a huge threat to the working face production and workers safety. The rib spalling scene of LW12301 during first weighting is shown in Figure 3.
GRADUAL DETERIORATION CHARACTERISTICS AND MECHANICAL CONDITION ANALYSIS OF RIB SPALLING DUE TO LARGE MINING HEIGHT AT FULLY-MECHANIISED WORKING FACE

Gradual deterioration characteristics of longwall face spall

Rib deterioration due to the large mining height at the working face is a complex mechanical process. The coal rib is not only influenced by the mining induced and tectonic stress, but also affected by some constrained effects of the deeper coal mass. From the perspective of fracture mechanics, the delamination fracture will occur when the tension or relative displacement between rib and deep coal mass reaches a certain limit. The deformation of the inelastic rib bearing segment continues to grow and the internal fissure damage within the ribside will accumulate. If the delamination fractures develop within the rib to inter-permeation, it is easy to reach rib instability in a large area. The interior coal fractures, under compressive stress, can rapidly expand causing rib failure along the parallel or biased maximum principal stress direction as shown in Figure 4. Rib failure can be simplified using the ‘thin plate’ model as shown in Figure 5. Rib gradual deterioration damage characteristics are shown in Figure 6.

Figure 2: Roadway arrangement of LW 12301

Figure 3: Rib spalling of LW12301 during first weighting

Figure 4: Rupture direction of crack tip under cyclic compressive load
Mechanical condition analysis of rib spalling in fully-mechanized working face

The rib of a large mining height working face bears the horizontal pressure from the front coal mass and the gravity stress of the overburden roof. The rib can be regarded as the statically indeterminate uniform beam (one end is clamped as support and the other end is simply supported). The rib geometry can be simplified to analyse the rib deflection produced in a horizontal direction. The gravity stress and the compression deformation in a vertical direction are ignored. The simplified model of the large mining working face rib is shown in Figure 7.

In Figure 7 $q$ is the horizontal load, $F_f$ is the friction resistance between coal seam and roof, $P$ is the roof pressure, $l$ is coal seam mining height, the rectangular coordinate system is built with the point $o$ as the origin of coordinates, $x$ is the vertical downward axis and $y$ is the horizontal axis positive to the right. The simplified model of the large mining height rib is mechanically analysed as follows:
For any cross section the centroid moment, the bending moment and deflection can be obtained by the following equations:

\[ M = \frac{3qlx - qx^2}{8} \]

\[ \omega = \frac{qx^2}{2EI} - \frac{3qlx}{8EI} \]  

(1)

(2)

Integrating the equation (2), the following is obtained:

\[ \omega = \frac{qx^3}{6EI} - \frac{3qlx^2}{16EI} + c_1 \]  

(3)

According to the characteristics of a cantilever beam, \( \omega = 0 \) when \( x = l \). Substituting it into equation (3), the following equation is obtained:

\[ c_1 = \frac{ql^3}{48EI} \]  

(4)

Integrating equation (3), the following equation is obtained:

\[ \omega = \frac{qx^4}{24EI} - \frac{qlx^3}{16EI} + \frac{ql^3x}{48EI} + c_2 \]  

(5)

According to the characteristics of a uniform beam, \( \omega = 0 \) when \( x = l \). Substituting it into equation (5), the following equation is obtained:

\[ c_2 = 0 \]  

(6)

Then the deflection curve equation is as follows:

\[ \omega = \frac{qx^4}{24EI} - \frac{qlx^3}{16EI} + \frac{ql^3x}{48EI} \]  

(7)

The maximum rib deflection can be obtained from equation (7). Differentiating equation (7) with respect to \( x \) and setting the derivative to zero yields the following equation:

\[ \frac{qx^3}{6EI} - \frac{3qlx^2}{16EI} + \frac{ql^3}{48EI} = 0 \]  

(8)

The stagnation points are therefore achieved from equation (8) as follow:

\[ x_1 = l \]  

(9)

\[ x_2 = \frac{1 + \sqrt{33}}{16} l \]  

(10)

\[ x_3 = \frac{1 - \sqrt{33}}{16} l \]  

(11)
Both the equation (9) and (11) cannot achieve the maximum value of deflection due to \( x_2 \subset [0,l] \), and finally the maximum is obtained when \( x_2 = \frac{1 + \sqrt{33}}{16} l \), at the position \( 0.422l \) from the roof and 0.578 times of the mining height, the maximum deflection is followed:

\[
\alpha_{\text{max}} = \frac{13ql^3}{2400EI}
\]

(12)

THE RESEARCH OF LW 12301 RIB SPALLING DEPTH

Theoretical analysis of rib spalling depth

According to the statistical analysis of the damage form and depth of rib spalling of different working faces in the Shendong mining area (including the Shangwan coal mine, Bulianta coal mine, Daliuta coal mine and Buertai coal mine). The authors established the relationship between the depth of rib spalling and mining height under similar conditions, as shown in Table.1.

Table 1: Mining height and the depth of rib spalling of five fully-mechanized working faces in Shendong mining area

<table>
<thead>
<tr>
<th>Name of working face</th>
<th>Mining height /m</th>
<th>Form of rib spalling</th>
<th>Depth of rib spalling /m</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW 12307 in Shangwan coal mine</td>
<td>3.5</td>
<td>tension crack failure</td>
<td>0.83</td>
</tr>
<tr>
<td>LW 12302 in Shangwan coal mine</td>
<td>4.5</td>
<td>tension crack failure</td>
<td>0.95</td>
</tr>
<tr>
<td>LW 22307 in Bulianta coal mine</td>
<td>6.8</td>
<td>tension crack failure</td>
<td>1.45</td>
</tr>
<tr>
<td>LW 52302 in Daliuta coal mine</td>
<td>6.6</td>
<td>tension crack failure</td>
<td>1.35</td>
</tr>
<tr>
<td>LW 42104 in Buertai coal mine</td>
<td>3.9</td>
<td>tension crack failure</td>
<td>0.45</td>
</tr>
</tbody>
</table>

The depth of rib spalling is associated with the rib spalling due to the tension cracking and direction of advance. Some researchers have determined the theoretical equation of rib spalling depth for tension crack failure in flat seams:

\[
\Delta P = M \cdot \tan(90^\circ - \varphi)
\]

(13)

Where, \( \Delta P \) is the depth of rib spall, \( M \) is the mining height, \( \alpha \) is the coal seam dip angle, \( \varphi \) is the coal seam friction angle.

Equation (13) is appropriately applied to working faces of which the mining height is below 3.0m.

Since Equation (13) is no longer adapted to the large mining height fully-mechanized working face in Shendong mining area, the correction coefficient \( K \) is introduced and a new equation which is suitable for large mining height working faces in Shendong mining area is obtained.

\[
\Delta P = K \cdot M \cdot \tan(90^\circ - \varphi)
\]

(14)

Substituting the data in Table. 1 into equation (14), the rib spalling correction coefficient \( K \) can be obtained. \( K \) ranges from 0.11 to 0.18, and thus the maximum depth of LW 12301 ranges from 0.98m to 1.61m.
Numerical simulation of rib spall in high working face

In order to analyse the rib spalling of LW12301, PFC2D software was used to carry out the numerical simulation. The width of the model was 400 m and height is 80 m. The working coal face thickness was 6.0m and the dip angle was 0°. The geometric model as shown in Figure 8.

The stress constraint in front of LW12301 is shown in Figure 9. It is clearly observed that the peak abutment stress zone is 5.0m ahead of the working face. The scale of rib spalling is shown in Figure 10. With the working face advancing, the coal failure becomes tensile failure from shear failure and gradually develops inwards the rib which causes the instability of the coal wall, and the stable height is 4.5 m. In the comparatively stable area, the phenomenon of slight rib spalling occurs with the depth of rib spalling of 0.8 m. The more serious rib spall occurs 2.5 m below the roof level reaching 1.5 m in depth. The rib spall takes the shape of a triangle. The numerical simulation result is consistent with the theoretical result.

![Figure 8: Graph of numerical analysis model](image1)

![Figure 9: Peak abutment stress zone in front of LW12301](image2)

![Figure 10: Rib spalling scale of LW 12301](image3)

**OBSERVATION RESULTS ANALYSIS OF LW12301 RIB SPALLING**

During the excavation process of LW12301 in the Shangwan mine, the observation results of the rib spalling depth were statistically summarised and are shown in Fig.11 and.12. According to the field statistics, it was found that the advancing speed of the working face influenced the rib spalling. During the normal mining of LW12301, the depth of rib spalling firstly increased and then decreased and finally reached a stable level of 1.0m to 1.2m. During the periodic weighting of LW12301, the peak stress in front of the working face incresed and moved forward, aggrevating the rib spalling. The average depth of rib spall was 1.55m, and the maximum was 1.7m. The observation results are consistent with both the numerical simulations and the theoretical results.
CONCLUSIONS

The research reveals the gradual deterioration characteristics of the working longwall face. The thin plate mechanical model of working face and empirical equation of rib spalling were established. The deduced deflection equation of rib deformation is: \( \omega = \frac{q x^3}{6EI} - \frac{3g l x^2}{16EI} + \frac{q I^3}{48EI} \), showing that the instability position is just above the mid seam (0.578 times of the mining height). According to the theoretical calculation, the maximum depth of rib spalling is between 0.98m and 1.61m, and the rib spalling initiates at 2.53m below the roof.

In numerical simulation, with the working face advancing, coal wall failure mechanism changed from the shear failure to tension failure. The coal wall stability decreased, the serious rib spall occurred at 2.5 m below the roof of the working face, and the total spalling depth extended to 1.5 m taking shape of a triangle.

The field statistical analysis of the rib spalling depth were consistent with both the theoretical analysis and numerical simulation, which verifies the reliability of this study.

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REVIEW OF HORIZONTAL SURFACE MOVEMENTS DUE TO LONGWALL COAL MINING USING NUMERICAL MODELLING

James Barbato\textsuperscript{1}, Bruce Hebblewhite\textsuperscript{2}, Rudrajit Mitra\textsuperscript{3} and Ken Mills\textsuperscript{4}

Strain is an important parameter for assessing the potential for impacts on surface features due to longwall mine subsidence, but it is also one of the most difficult parameters to predict. Whilst profiles of strain can be highly variable and irregular, zones of net compression and net tension generally develop above longwalls. By considering the relative horizontal surface movements over longer bay lengths, they become more regular and, hence, more predictable. Numerical modelling has been undertaken using universal distinct element code (udec) to assist with the development of predictive methods for the relative horizontal movements over the various zones above an active longwall. The numerical modelling was used to assess the effects of varying surface topography on the horizontal movements, including slopes, scarps, hills and small valleys. Predictive equations have been developed for the net compression within the sagging curvature zone and the net openings within the hogging curvature zones. These equations are consistent with the findings from reviews of ground monitoring data in NSW coalfields.

BACKGROUND

The prediction of horizontal movement and strain due to longwall mine subsidence have historically been based on simple empirical relationships with mining geometry or other subsidence parameters. Whilst these empirical methods generally provide reasonable predictions of the regular (i.e. conventional) ground movements, they are often exceeded in discrete locations due to irregular or anomalous movements.

More recently, statistical methods have been used to predict the distribution of strain based on measured ground monitoring data. These methods provide the predicted probabilities of exceedance (i.e. confidence intervals) for strain based on both the regular and irregular movements. However, these statistical methods are often limited in use to locations where the mining geometry and overburden lithology are similar to the mining areas from which the monitoring data were collected.

Research is currently being undertaken to improve the predictive methods for horizontal movement and strain at the surface due to longwall coal mining. It has been found that by considering the relative horizontal movements over longer bay lengths, they are more regular and, hence, become more predictable when compared with those measured across the standard survey bay lengths.

It has also been identified by various authors that areas of sagging curvature tend to be net compressive zones and areas of hogging curvature tend to be net tensile zones above the active longwall. For this reason, the predictive methods for strain have been developed based on a two-step process. The relative horizontal movements are predicted across each of the zones above the active longwall and then the distributions of strain within each of these zones are predicted based on the net or overall movements.

The zones above a subcritical longwall and the idealised profiles of incremental vertical subsidence (S) and incremental horizontal movement (u) are illustrated in Figure 1. The incremental parameters are the additional ground movements due to the extraction of the active longwall within the series.

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The initial research has focused on the net compression across the sagging zone above the active longwall, referred to as ‘panel closure’ ($\Delta u_c$). This parameter is the difference between the maximum (i.e. most positive, $u_{\text{max}}$) and the minimum (i.e. most negative, $u_{\text{min}}$) horizontal movements transverse to the longwall, i.e. at right angles to the centreline of the active longwall. The research has also investigated the net tensions across the maingate and pillar zones, referred to as ‘maingate opening’ ($\Delta u_m$) and ‘pillar opening’ ($\Delta u_p$).

The relationships between panel closure and the mining geometry and other subsidence parameters were reviewed using the available ground monitoring data from the Southern Coalfield. It was identified that there is a strong relationship between the measured panel closure and the maximum incremental vertical subsidence, as illustrated in Figure 2, based on the available 3D monitoring lines from the Southern Coalfield.
It was found that, after an initial development of vertical subsidence up to around 100 mm, there are reasonably linear relationships between panel closure and vertical subsidence for each of the monitoring lines. Similar relationships were also found based on monitoring data from the Newcastle, Hunter and Gunnedah Coalfields, comprising a wide range of mining geometries (i.e. longwall widths, depths of cover and extraction heights) and overburden lithology. The relationships appear to be non-linear during the initial development of subsidence (i.e. less than around 100 mm) at which time the extraction face is either approaching or partially beneath the monitoring lines.

The gradients of the linear segments typically vary between 0.15 and 0.25, but are up to 0.35 for some cases. A review of the empirical data found that the gradients were generally towards the lower end of the range for the cases where the surface topography and seam were relatively flat and the ground subsided regularly with no localised or elevated irregular/anomalous strains. Conversely, the gradients were generally towards the upper end of the range for cases with incised topography, including steep slopes and small valleys, and for cases with irregular ground movements comprising localised and elevated strains.

The empirical data suggests that a simple predictive equation can be developed for panel closure based on a linear relationship with maximum incremental vertical subsidence, when the magnitude of subsidence is greater than about around 100 mm. The generalised equation for panel closure is:

\[ \Delta u_c = \alpha + (\beta + \delta) S_{\text{max,inc}} \]  

(1)

The first coefficient alpha (\(\alpha\)) represents the component of movement due to horizontal compression of the overburden strata, which is largely due to the redistribution of horizontal in situ stress. The second coefficient beta (\(\beta\)) represents the component of horizontal movement that is directly related to the vertical component, when the surface, overburden and seam are relatively uniform and flat and when the ground subsides regularly. The third coefficient delta (\(\delta\)) represents the component of horizontal movement due to topographical features (such as sloping terrain, scarps, hills and small valleys), sloping overburden strata, sloping seam and irregular ground movements.

**NUMERICAL MODELLING**

Numerical modelling was carried out to further investigate the relative horizontal movements across each of the zones above an active longwall and their relationships with maximum vertical subsidence and varying surface topography. The numerical modelling was undertaken using Universal Distinct Element Code (Itasca Consulting Group, Inc.). UDEC is a two-dimensional distinct element method that models the rock mass as an assembly of discrete elements that interact via compliant contacts or interfaces.

**Development of the generic numerical model**

A generic two-dimensional numerical model was initially developed for the Southern Coalfield based predominantly on the conditions around Appin and West Cliff Collieries. The initial model assumed a flat surface, seam and overburden strata. The discontinuities were modelled as a series of parallel horizontal and vertical joints.

The properties of the overburden were predominately based on the testing results from three boreholes at Appin and West Cliff Collieries (MacGregor and Conquest 2005), but also considered information available in literature and from previous UDEC numerical studies including CSIRO Petroleum (2002), Keilich (2009) and Zhang (2014).

The thickness of the strata layers adopted in the model are included in Table 1. These thickness values were based on the three boreholes, but were adjusted so that the total depth of cover above the Bulli
Seam was 500 metres. The generic model allows the longwall widths to be varied between 150 and 600 metres and, therefore, considers width-to-depth ratios ranging between 0.3 and 1.2.

There is a wide range of material and joint properties documented in literature. It was considered that no single value could be representative for each of these properties due to variations in the testing results, scaling effects, variations over depth and in plan and effects of inclusions and discontinuities. There is inherent uncertainty in these properties in situ, let alone when the strata are subjected to mine subsidence.

The generic numerical model was reviewed, therefore, based on a range of material and joint properties and then calibrated using ground monitoring data. The main variables considered in the initial calibration were the block size (i.e. discontinuity spacing), horizontal in situ stress, rock mass properties and discontinuity properties.

The calibration process considered three block sizes, referred to as Types B1, B2 and B3 (from smallest to largest). The spacing of the horizontal discontinuities varied between 9 to 15 metres for the sandstone formations and between 3 to 5 metres for the claystone formations. The ratio of the horizontal to vertical block dimensions was taken to be 1.5. The vertical discontinuities were staggered so that the blocks formed a ‘brick’ type pattern. The horizontal in situ stress was modelled as equal to the vertical stress (Type S1) and two times the vertical stress (Type S3).

Three groups of rock mass properties were considered in the initial calibration, referred to as Types M1, M2 and M3 (from weakest and most flexible through to strongest and stiffest). The rock mass properties at the upper end of the range (i.e. Type M3) were based on the mean of the test results of the 48 core samples taken from the three boreholes at Appin and West Cliff Collieries (MacGregor and Conquest 2005), i.e. no reduction for scale. The properties at the lower end of the range (i.e. Type M1) were based on the reduction for scale proposed by McNally (1996) and represented values around one-standard deviation below the mean of the test results.

There is a wide range of discontinuity properties documented in literature and adopted in the previous UDEC modelling studies. The available information in literature is predominantly for intact rock and there is little guidance on the appropriate properties when the joints are subjected to mine subsidence movements. The models considered a range of joint properties, based on reviews of the available information, and are referred to as Types J1, J2 and J3 (from weakest and most flexible through to strongest and stiffest).

The joint normal stiffness (jkn) for Sydney sandstone and shale provided by Bertuzzi and Pells (2002) range between 10 GPa/m (5 to 10 mm thickness) and 4000 GPa/m (tight) for major erosional bedding planes, and between 500 GPa/m (3 mm thickness) and 4000 GPa/m (tight) for joints. The joint normal stiffness adopted in the previous UDEC modelling studies in the Southern Coalfield were 3000 GPa/m (CSIRO Petroleum 2002), between 21 and 204 GPa/m (Keilich, 2009) and 26 GPa/m (Zhang 2014).

An initial review of the numerical models found that joint normal stiffness towards the lower end of the range of those documented in literature provide results that more closely match the available ground monitoring data. The lower stiffness is likely to reflect the large scale post peak deformation (i.e. horizontal shear and vertical dilation) that develop as a result of mine subsidence. The range of the joint normal stiffness adopted in the generic numerical models was between 10 GPa/m (Type J1) and 50 GPa/m (Type J3). The joint shear stiffness (jks) was taken to be one-tenth of the normal stiffness, as adopted in the previous UDEC numerical modelling studies.

The peak cohesions (C) for the discontinuities considered in the initial models ranged between 1.5 MPa and 6.25 MPa for the sandstone units and between 1.0 MPa and 3.5 MPa for the claystone units. The joint cohesions represented values up to around one-quarter of the tested cohesions for the rock mass. The peak friction angles (ϕ) for the discontinuities ranged between 22 and 27° for the sandstone units.
and between 20 and 26° for the claystone units. The residual cohesions ($C_{res}$) and residual friction angles ($\phi_{res}$) were taken to be 60% of their peak values.

The tensile strengths of the discontinuities (T) were taken to be zero. A dilation angle of 5° was adopted for all horizontal and vertical discontinuities. A Coulomb slip with residual strength model was adopted for the horizontal and vertical discontinuities.

**Calibration of the generic numerical model**

The numerical model was initially calibrated by comparing the modelled maximum vertical subsidence with those provided by empirical prediction curves. The calibration process considered 378 models based on combinations of the three block sizes (B1, B2 and B3), two levels of horizontal in situ stress (S1 and S3), three groups of material properties (M1, M2 and M3), three groups of joint properties (J1, J2 and J3) and seven longwall widths of 150, 225, 300, 375, 450, 525 and 600 metres.

A template text (*.txt) file was developed which could be ‘called’ into UDEC to generate the numerical models based on each of the block sizes, horizontal in situ stresses, material properties and joint properties. The models could be batched allowing each of the 378 models to be run in succession.

The modelled maximum vertical subsidence divided by the seam thickness versus the longwall width-to-depth ratio are illustrated in Figure 3. The empirical prediction curves for a single isolated longwall based on the Incremental Profile Method (IPM) (Waddington and Kay 1995) and Holla and Barclay (2000) are also shown for comparison in both the graphs provided in this figure.

![Figure 3: Maximum predicted vertical subsidence divided by seam thickness versus width-to-depth ratio for a single isolated longwall](image)

The range of results obtained from the 378 numerical models are shown on the left side of Figure 3. The model that matched the empirical prediction curves the closest is shown as the green curve on the right side of this figure. This model was based on Block Type B2, Material Type M1 and Joint Type J2 and was adopted as the ‘base case’, i.e. the standard model with a flat surface and seam.

A summary of the adopted strata thickness and rock mass properties (i.e. Type M1) for the base case numerical model is provided in Table 1. These properties were based on MacGregor and Conquest (2005) and modified for scale based on McNally (1996). A summary of the joint properties adopted in the base case numerical model (i.e. Type J2) is provided in

**Table 2.** The joint normal stiffness ($jkn$) is 30 GPa/m and joint shear stiffness ($jks$) is 3 GPa/m in the base case numerical model.
Table 1: Strata thicknesses and rock mass properties adopted in the base case model

<table>
<thead>
<tr>
<th>Unit</th>
<th>Thickness (m)</th>
<th>$p$ (kg/m$^2$)</th>
<th>$K$ (GPa)</th>
<th>$G$ (GPa)</th>
<th>$C$ (MPa)</th>
<th>$\phi$ (deg)</th>
<th>$T$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>150</td>
<td>2400</td>
<td>3.33</td>
<td>2.00</td>
<td>7.0</td>
<td>34</td>
<td>0.5</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>20</td>
<td>2400</td>
<td>3.45</td>
<td>2.48</td>
<td>4.0</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>30</td>
<td>2700</td>
<td>5.00</td>
<td>2.31</td>
<td>6.0</td>
<td>25</td>
<td>0.5</td>
</tr>
<tr>
<td>Bulga Sandstone</td>
<td>200</td>
<td>2500</td>
<td>5.56</td>
<td>4.17</td>
<td>10.0</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>10</td>
<td>2700</td>
<td>6.17</td>
<td>4.07</td>
<td>9.0</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>30</td>
<td>2700</td>
<td>7.47</td>
<td>5.37</td>
<td>7.0</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Wombbarra Claystone</td>
<td>30</td>
<td>2600</td>
<td>6.90</td>
<td>4.96</td>
<td>10.0</td>
<td>25</td>
<td>0.5</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>30</td>
<td>2600</td>
<td>7.78</td>
<td>5.63</td>
<td>12.0</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Bulli Coal</td>
<td>3</td>
<td>1500</td>
<td>1.54</td>
<td>0.97</td>
<td>2.0</td>
<td>25</td>
<td>0.5</td>
</tr>
<tr>
<td>Sub-Bulli Strata</td>
<td>-</td>
<td>2500</td>
<td>8.00</td>
<td>4.80</td>
<td>15.0</td>
<td>25</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2: Joint properties adopted in the base case model

<table>
<thead>
<tr>
<th>Unit</th>
<th>$C$ (MPa)</th>
<th>$\phi$ (deg)</th>
<th>$C_{v0s}$ (MPa)</th>
<th>$\phi_{v0s}$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>2.50</td>
<td>24.8</td>
<td>1.60</td>
<td>14.9</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>2.26</td>
<td>24.0</td>
<td>1.35</td>
<td>14.4</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>2.75</td>
<td>21.2</td>
<td>1.65</td>
<td>12.7</td>
</tr>
<tr>
<td>Bulga Sandstone</td>
<td>4.50</td>
<td>24.0</td>
<td>2.70</td>
<td>14.4</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>2.75</td>
<td>24.0</td>
<td>1.65</td>
<td>14.4</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>3.25</td>
<td>26.0</td>
<td>1.95</td>
<td>15.6</td>
</tr>
<tr>
<td>Wombbarra Claystone</td>
<td>3.00</td>
<td>22.0</td>
<td>1.80</td>
<td>13.2</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>4.75</td>
<td>23.2</td>
<td>2.65</td>
<td>13.9</td>
</tr>
<tr>
<td>Sub-Bulli Strata</td>
<td>4.25</td>
<td>22.0</td>
<td>2.55</td>
<td>13.2</td>
</tr>
</tbody>
</table>

The base case numerical model was reviewed using ground monitoring data from the Southern Coalfield as described in the following section.

**Review of the base case numerical model**

The results obtained from the ‘base case’ numerical model were compared with the ground movements measured along a number of monitoring lines at Appin and West Cliff Collieries. Examples are provided in Figure 4 for the M-Line in Area 3 at Appin Colliery, Figure 5 for the ARTC, HW2 East and HW2 West Lines in Area 7 at Appin Colliery, and Figure 6 for the B-Line in Area 5 at West Cliff Colliery.

Figure 4: Measured versus predicted vertical subsidence and horizontal movement along the M-Line in Area 3 at Appin Colliery
The modelled vertical subsidence profiles in Appin Area 3 (refer Figure 4) reasonably match those measured along the M-Line. The observed subsidence adjacent to the tailgate of LW301 (i.e. left side of figure) was less than that obtained from the numerical model, however, this area was affected by a large valley closure movement across a nearby stream. There is a small lateral shift between the measured and predicted profiles above the maingate of LW302 (i.e. right side of longwall). This lateral shift could possibly be due to the influence of the sloping terrain or the proximity of the Nepean River valley.
The shapes of the modelled horizontal movement profiles in Appin Area 3 are reasonably similar to those observed above the longwalls. There is, however, a relatively uniform difference in magnitude (i.e. vertical shift) that appears to be largely due to the valley closure movement at the nearby stream. The modelled total panel closure above the longwalls of 0.16 metres is less than the measured total panel closure of 0.21 metres. The difference between observed and predicted panel closure could be partly the result of the influence of the sloping terrain or the proximity of the Nepean River valley.

The modelled vertical subsidence profiles in Appin Area 7 (refer Figure 5) reasonably match those measured along the ARTC, HW2 East and HW2 West Lines. The maximum predicted vertical subsidence movements above the longwalls are similar to the maxima observed. However, the minimum predicted vertical subsidence above the chain pillars are less than those observed.

The modelled horizontal movement profiles differ in shape and magnitude when compared with those measured in Appin Area 7. The observed horizontal movements in this area, however, were largely influenced by the sloping terrain, valley closure movements due to the creeks above the longwalls and the nearby Nepean River valley, and non-conventional ground movements due to near surface geological structures. This case study highlights the complexity of predicting horizontal movements where they are influenced by topographical and geological features. The surface slope, streams and near surface geological structures were not considered in the base case numerical model.

The modelled vertical subsidence profiles in West Cliff Area 5 (refer Figure 6) reasonably match those measured along the B-Line. The magnitudes of vertical subsidence obtained from the numerical model were less than the maxima measured. It is noted, however, that the vertical subsidence along the B-Line was greater than that measured along the J-Line, which was located above the opposite ends of these longwalls. This may indicate that increased vertical movements developed along the B-Line, possibly due to the influence of the nearby Georges River valley, or the non-conventional ground movements resulting from near surface geological structures.

The shape of the modelled total horizontal movement profile (i.e. solid red line) in West Cliff Area 5 is reasonably similar to that observed (i.e. solid blue line). The shapes and the magnitudes of the modelled incremental horizontal movement profiles (i.e. dashed red lines) differ from those observed (i.e. dashed blue lines). The main differences occur adjacent to the longwall main gates, where locally increased horizontal movements were observed. These differences could be partly the result of the non-conventional ground movements that developed along this monitoring line.

It has been considered that the base case numerical model provides reasonable predictions of vertical subsidence and horizontal movements based on the comparisons with the monitoring data. The three case studies using the monitoring data from Appin and West Cliff Collieries, however, highlight that the modelled horizontal movements obtained using the base case model (i.e. uniform surface, seam and overburden) can differ greatly from those observed. These differences were primarily due to the influence of the surface topography (i.e. sloping terrain and valleys) and non-conventional ground movements resulting from near surface geological structures.

The base case numerical model has been further refined to include varying surface topography in order to review its influence on the horizontal movements resulting from longwall mining.

**Review of varying surface topography using the UDEC numerical model**

Several numerical models have been developed to review the effects of sloping terrain, scarps, hills and small valleys. An example of the model with sloping terrain is illustrated on the left side of Figure 7. The profiles of vertical subsidence and horizontal movement obtained from the numerical model based on varying surface slopes are shown on the right side of this figure.
The numerical models indicate that an increasing surface slope results in greater vertical subsidence on the upslope side and lower vertical subsidence on the downslope side. The surface slopes also result in greater panel closure (i.e. differential horizontal movement above the longwall) and increased net horizontal movement in the downslope direction. These results are consistent with observations from the NSW Coalfields.

The ratios of the incremental panel closure to incremental vertical subsidence obtained from the numerical models are illustrated in Figure 8 for sloping terrain (left side) and scarps (right side). The different curves shown on the left and right sides of this figure are for longwall widths varying between 300 and 600 metres, i.e. width-to-depth ratios varying between 0.6 and 1.2, based on an average depth of cover of 500 metres.

It can also be seen from Figure 8, that the ratio of panel closure to maximum vertical subsidence increases as the surface slope or the scarp height increases. The relationship between panel closure and maximum vertical subsidence is reasonably consistent over the range of longwall widths considered in the numerical analysis. The component of panel closure due to the presence of a slope or scarp can be represented by a term based on the change in elevation ($\Delta z$) across the width of the longwall, i.e. $\delta \cdot (\Delta z)^2$. The coefficient delta ($\zeta$) has a value less than one, as the curves illustrated in Figure 8 have slight curvatures, with the gradients decreasing with increasing slope or scarp height.
The generalised predictive equation for panel closure is provided in Equation 2. This equation has been developed based on the review of the ground monitoring data and the numerical modelling.

\[ \Delta u_c = \alpha_c W + \eta_c \left[ \beta_c + \delta_c \left( \frac{\Delta z}{W} \right) \right] s_{\text{max,inc}} \]  

(2)

The coefficient alpha \( (\alpha_c) \) and the variable for the change in surface elevation \( (\Delta z) \) have been non-dimensionalised based on the longwall void width \( (W) \). The change in elevation across the width of the active longwall is illustrated in Figure 9 based on a slope (left side) and a scarp (right side).

\[ \Delta u_c = \alpha_c W + \eta_c \left[ \beta_c + \delta_c \left( \frac{\Delta z}{W} \right) \right] s_{\text{max,inc}} \]

\( \Delta \ u_c \) = \( \alpha_c \) \( W \) + \( \eta_c \) \( [\beta_c + \delta_c \left( \frac{\Delta z}{W} \right) ] s_{\text{max,inc}} \)

\[ \Delta z \]

\[ W \]

\[ \Delta u_c \]

\[ \alpha_c \]

\[ \beta_c \]

\[ \delta_c \]

\[ \eta_c \]

\[ s_{\text{max,inc}} \]

Figure 9: Change in elevation due to a slope (left side) and scarp (right side)

The coefficient eta \( (\eta_c) \) is the localisation factor to account for the higher panel closures observed when irregular ground movements develop, due to the presence of small valleys or due to anomalies. The value varies between 1.0 when the ground subsides regularly and 1.6 when irregular compressive strains in the order of 5 and 6 mm/m develop.

The preliminary coefficients for the panel closure predictive equation for the Southern Coalfield, derived from the available monitoring data and the numerical models, are summarised in Table 3. The terms ‘up slope’ and ‘up step’ refer to the cases where the surface elevation above the maingate is greater than that above the tailgate and vice versa for ‘down slope’ or ‘down step’.

![Table 3: Preliminary coefficients for panel closure for the Southern Coalfield](image)

<table>
<thead>
<tr>
<th>( \alpha_c )</th>
<th>( \beta_c )</th>
<th>Case</th>
<th>Type</th>
<th>( \delta_c )</th>
<th>( \zeta_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 ~ 0.4 \times 10^{-3}</td>
<td>0.11</td>
<td>Slope</td>
<td>Up slope</td>
<td>0.60</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Down slope</td>
<td>0.20</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Scarp</td>
<td>Up step</td>
<td>1.50</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Down Step</td>
<td>1.20</td>
<td>0.8</td>
</tr>
</tbody>
</table>

The predictive equation for panel closure can be used in other mining areas outside the Southern Coalfield by modifying the coefficients to suit the local conditions. This can be achieved by reviewing the available ground monitoring data to establish the relationship between panel closure and vertical subsidence \( (\alpha_c \text{ and } \beta_c) \) and the effects of topography \( (\delta_c \text{ and } \zeta_c) \) and irregular movements \( (\eta_c) \).

CONCLUSIONS

Predictive equations have been developed for the relative horizontal movements across various zones above an active longwall based on a review of ground monitoring data and numerical modelling using UDEC. The predictive equation for panel closure in Equation 2 represents the maximum or net compression above the active longwall.

The numerical modelling was used to extend the preliminary equation developed from the empirical review to include the influence of varying surface topography. Further numerical modelling will be undertaken to assess the influence of the near surface lithology and geological structure.

The predictive equations for panel closure, maingate opening and pillar opening are being used as the basis for the ongoing research into the development of the predictive equations for strain.
REFERENCES


INSEAM BOREHOLES TO AND BEYOND 2000 M WITH A COMBINATION OF SLIDE AND ROTARY DRILLING

Frank Hungerford¹ and Wayne Green²

ABSTRACT: Directional drilling has been the established form of in-seam drilling for gas drainage, exploration and water management for the past two decades. Although there has been a desire to achieve longer boreholes to depths similar to that achieved with surface drilling, seam conditions, equipment capacity and drilling methods have limited in-seam drilling depths. Development into a new area of Metropolitan Colliery required boreholes to depths of 2000 m to provide the required gas drainage. This offered an opportunity to use a combination of slide and rotary drilling similar to that used with Surface to Inseam (SIS) drilling to achieve the required depths. This paper describes the drilling techniques used and presents the results of the drilling.

INTRODUCTION

Metropolitan Colliery is developing in the Bulli seam into a new area which has high gas content; carbon dioxide being the dominant gas. Limited access does not allow the standard gas drainage drilling program to be employed to drain the gas prior to mining. With the proposed gate-roads 2000 m long, the colliery approached VLI to attempt drilling long, in-seam boreholes to 2000 m and beyond to provide drainage coverage. Drilling shorter holes would necessitate a staged and disrupted development to allow progressive drilling and gas drainage.

Directional drilling in coal mining has been developed to a stage where standard practices allow boreholes to 1400 m to be drilled regularly in slide drilling mode with the occasional borehole being drilled to beyond 1700 m. The record depth for inseam boreholes was 1761 m in 2002 in Australia (Valley Longwall 2002). To extend the depth to 2000 m, a combination of directional slide and rotary drilling was planned to be applied from the start. Slide drilling mode involves feeding the Down Hole Motor (DHM) into a borehole with “flip-flopping” orientations to provide directional control while with rotary drilling mode, the drill string is rotated over extended lengths while the desired trajectory and alignment are maintained. This paper presents the results of the successful progress with that drilling.

DEPTH LIMITATIONS

In achieving a depth of 1005 m with the earliest NQ size configuration, penetration had started surging beyond 60 m so the penetration rate was progressively reduced to prevent stalling of the DHM as borehole depths increased (Hungerford et al., 1988). Eventually surging and the resultant stalling would cause the termination and limit the depth of longholes. A 2-7/8” Accu-dril DHM was offered to the industry in 1992 through Asahi (Walsh and Hungerford, 1993.). This unit had a non-magnetic, high-torque, low-speed 4-5 lobe motor section (Hungerford 1995) which, when fitted with a 1.25°bend and combined with a 96.1 mm diameter Poly Crystalline Drill (PCD) bit, greatly reduced surging (which had been attributed to in-hole friction) and drilling rates improved. In 1993 and 1994, the first two boreholes drilled with this configuration achieved lengths of 1233 m and 1535 m (Walsh and Hungerford 1993). This configuration was established as the standard for in-seam drilling in Australia and eventually the world.

With the higher thrust loading involved, the capacity of in-hole equipment was going to be tested. Analysis of drilling data collected from long-holes with torque/drag models established, showed that the NQ drill rod strength was adequate for the depths achieved to date (Gray 1991), but that borehole depth would eventually be limited due to helical buckling with the current drilling techniques (Gray 1992). Tests of rod strength had proven that the preferred drill rod joint know as CHD being adopted by the industry was the superior rod in strength and ease of handling in jointing (Gray and Daniel 2000). Withdrawal friction and rotation were also thought to be limiting factors from these analyses.

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² Technical Services Superintendent, Metropolitan Colliery, Peabody Energy
The severity and number of bends in the initial stages of a borehole influences the rate of drilling. The in-hole friction increases as borehole depth increases (Hungerford et al., 2012). Any attempt at longhole drilling may necessitate limiting bends in the borehole, particularly in the initial stages.

**Surveying**

The survey systems were thought to be a limiting factor with the DDM-MECCA initially preferred over the DGS due to signal strength. Previous boreholes drilled to and beyond 1500 m had suffered from signal strength problems when using the DGS. Subsequent development of the Drilling Guidance System (DGS) (McCabe and Hellyer 2013) apparently improved signal strength and transmission but this yet to be proved over the longer lengths.

To enhance the chances of successful signal transmission at depth, relatively new CHD rods were used with the DPI RCS (Rod Communication System – similar to the AMT MECCA) installed.

**Drilling configuration**

The DPI equivalent of the non-magnetic 4/5 Accu-Dril DHM was used with a 1.125° bent housing fitted with a 1 mm thick wear pad. A standard Asahi 96.1 mm diameter PCD bit was used which combined for an off-set at the bit (B) in Figure 1 of 6.7 mm. This was equivalent to being fitted with a bent housing of 1.22°. In initial rotation of the DHM, the heel of the bend would be flexed 2.9 mm to fit within the 96.1 mm diameter until the hole diameter was increased by the rotation.

After the first hole, the bit size was increased to 99 mm by moving the outer cutters outward. This reduced the odd-set at the bit (B) to 5.3 mm and thus reduced the effective bend to 1.12°. In rotating the DHM, the heel of the bend fits within the 99 mm diameter thus avoiding any flexing of the DHM.

**Figure 1: Deflection of DHM (A) and bit (B) with wear pad**

**RIG CAPACITY**

Due to a combination of size limitations in getting equipment into the mine and availability of drill rigs, the initial drill supplied for the project was a modular VLI Series 1000. This drill rig had a thrust capacity of 104.6 kN compared to 140 kN of the track mounted Series 1000. The lesser thrust capacity was possibly a limitation on achieving 2000 m when compared to the 220 kN capacity of the Fletcher LHD used previously for the record drilling to 1761 m.

**Drilling Practice**

Longhole drilling had become an established practice with the conventional flip-flopping of DHM bend orientation for directional control. But with the practice of a change in orientation every 6 m, 1200 m long boreholes were relatively common with only the occasional borehole being drilling beyond 1500 m. Several methods had been employed to extend boreholes depths. These included:

- Reaming sections of the borehole to a larger diameter (Valley Longwall 2002),
- Reducing the bend on the DHM, using an impregnated bit and high speed 1/2 lobe DHM (Kravits et al., 1999),
- Increasing the length drilled on each side of the flip-flop drilling method (Gray 1991), and
- Employing a rotary/slide method of drilling commonly used by SIS drilling and previously in some underground drilling operations (Eade 2002).

Before drilling commenced, the drillers were instructed on the drilling practices required for the project. Most drillers had used rotary slide for short sections of drilling on previous projects so were conversant with the practice. The initial drilling parameters included:

- Slide drilling to establish position and dip within the seam and on alignment/azimuth.
- Slide drilling to maintain lateral borehole curve to remain on line.
- Slide drilling to target the seam roof for seam profile definition.
- Slide drilling to establish each branch.
- 200 litres/min water flow to operate the DHM.
- Rotary drilling whenever possible when comfortably on line.
- Rotary drilling at 30 – 60 rpm to limit damage and wear to the DHM.
- Record usual drilling parameters of thrust and hold-back hydraulic pressures and water idle and drilling pressures.
- Survey at 6 m intervals and also record each 3 m intermediate survey.
- Record main hydraulic pump pressure when rotary drilling.

To manage all returns from the borehole, drilling was to be through a 150 mm standpipe and valve. With high gas flows expected from the boreholes, the rig was set back from the face to allow a 3 m enclosure (Figure 2) for withdrawing the DHM from the hole.

**Figure 2: Standpipe configuration with 3 m enclosure**

### Drilling Conditions

Ultimately, drilling conditions have an influence on the borehole depths achieved. Good intact coal conditions allow for easy directional drilling with DHMs with minimum problems with in-hole collapse and bogging. If any unstable conditions are experienced, ongoing drilling beyond that point will always be suspect with loss of expensive equipment being the main concern. Drilling to extreme depths beyond the 600-700 m over-coring capacity eliminates over-coring as insurance and plans need to be in place to eventually recover the equipment when intersected. The Bulli seam has an average thickness of 3.0 m with no geological structures expected in the area of the proposed drilling.

### Drilling Results

Nine boreholes were completed from two drill sites at the time of writing (Table 1). All boreholes were drilled with a combination of slide and rotary drilling. Each borehole had different applications of rotary drilling, off-set entry angle and eventual lateral deviation. That delivered a different depth in each borehole from which slide drilling could no longer continue and drilling was continued with rotary drilling only. The table also indicates the depth to which slide drilling was possible, the lateral deviation and the reason for terminating each borehole.

Figure 3 shows eight of the nine boreholes plotted on the mine plan showing the proposed gate-road development. Drilling conditions were found to be stable with no structures or boggy conditions experienced.
Table 1: Borehole Data

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Date</th>
<th>Depth (m)</th>
<th>Slide to (m)</th>
<th>Lat Dev (m)</th>
<th>Terminated</th>
</tr>
</thead>
<tbody>
<tr>
<td>EX03</td>
<td>15/06/2015</td>
<td>1716</td>
<td>1746</td>
<td>116 L</td>
<td>No signal</td>
</tr>
<tr>
<td>EX02</td>
<td>19/07/2015</td>
<td>1875</td>
<td>1851</td>
<td>58 L</td>
<td>Floor</td>
</tr>
<tr>
<td>DH01</td>
<td>28/07/2015</td>
<td>1971</td>
<td>1803</td>
<td>31 L</td>
<td>Floor</td>
</tr>
<tr>
<td>DH04</td>
<td>9/08/2015</td>
<td>2001</td>
<td>1821</td>
<td>129 L</td>
<td>Roof</td>
</tr>
<tr>
<td>DH05</td>
<td>31/08/2015</td>
<td>2007</td>
<td>1653</td>
<td>78 R</td>
<td>To design</td>
</tr>
<tr>
<td>DH08</td>
<td>23/09/2015</td>
<td>2151</td>
<td>1743</td>
<td>40 L</td>
<td>No rods</td>
</tr>
<tr>
<td>DH09</td>
<td>7/10/2015</td>
<td>2103</td>
<td>1761</td>
<td>83 L</td>
<td>No rods</td>
</tr>
<tr>
<td>DH10</td>
<td>27/10/2015</td>
<td>2007</td>
<td>1761</td>
<td>121 L</td>
<td>To design</td>
</tr>
<tr>
<td>DH11</td>
<td>17/11/2015</td>
<td>2016</td>
<td>1920</td>
<td>166 L</td>
<td>No rods</td>
</tr>
</tbody>
</table>

Figure 3: Plan of longhole coverage of proposed gate-roads

Because of limited access, the boreholes could not be designed as straight holes along their target azimuth with a zero lateral deviation. In that, they were designed with lateral curves to provide the required drainage coverage and not set up specifically to create depth records. The lateral deviation is plotted for the boreholes from the two sites (Figures 4 and 5).

With little geological and RL information in the area of the proposed drilling, the initial borehole (EX03) served as an exploration hole with regular roof intersections to define the seam profile. The borehole and seam profile (Figure 6) also shows boreholes crossed and the expected location of cut-troughs in future gate-road development. This borehole was terminated with survey signal problems at 1716 m, which had established a new world record for underground drilling.

Each subsequent borehole increased that record until DH08 established the world record at 2151 m (Table 1). Being the first borehole from the 9 c/t site, regular roof intersections were completed for seam profile definition (Figure 7). The borehole was drilled with a combination of slide and rotary out to 1743 m (Table 1, Figure 8); at which point 45% had been slide mode with 55% rotary. The remainder of the borehole was drilled wasin rotary mode.
Figure 4: Lateral Deviation of boreholes from 6c/t site

Figure 5: Lateral Deviation of boreholes from 9c/t site

Figure 6: Borehole Profile Metro EX03
The plot of thrust on the drill string for slide drilling (Figure 9) displays the usual trend of increased drilling rate increase with depth, indicating the increased friction effect of curves earlier in the borehole (Hungerford et al., 2012). This trend extrapolated to 140 kN thrust would indicate the greater capacity track mounted Series 1000 rig would possibly have managed slide mode drilling to 1900 m in this borehole.

In rotary drilling mode, in-hole friction is greatly reduced (Figure 9) and only starts to increase gradually from the 1400 m depth.

This reduction in friction also provided consistent feed at the bit compared to the surging feed experienced in slide mode. With consistent loading on the bit and DHM, drilling rates are more consistent over depth compared to the rapid reduction in the slide drilling rate (Figure 10) to avoid stalling the DHM.
Water pressure was a concern before drilling commenced with the increase in idle pressure with drill string length likely to approach the maximum available from the pump. The drilling was commenced using 200 litres/min water flow to assess the progressive increase with depth and identify any potential problems by extrapolating that trend to beyond 2000 m. From Figure 11, the idle pressure increased from 1.5 MPa (at the start) at a rate of 0.15 MPa/100 m. The drilling pressure decreased gradually over the depth of the borehole as drilling rates decreased.

Although the idle pressure was more than 2 MPa below the maximum available pump pressure at depths beyond 2000 m, problems were encountered when starting the DHM. The pressure spike (on starting) occasionally took the water pressure above 7 MPa and stalled the pump before the DHM started. Lower water flow (generating lower pressure) was usually required to start the pump for drilling before the flow was increased back to 200 litres/min.
Figure 11: Vertical response curve over 3 m intervals

The usual assessment of DHM steering response has been over 6 m intervals to match the usual 6 m flip-flop drilling method (Hungerford et al., 2012). With 6 m intervals unlikely to be used with regular use of rotary drilling, the intermediate 3 m surveys allowed the vertical and lateral response curves to be assessed over 3 m intervals (Figures 12 and 13). These plots are in the order of 50% that established for 6 m intervals with a standard 1.25° bend and 96.1 mm bit configuration. The magnitude of deviations with a 1.12° equivalent bend should be slightly less than half over 3 m.

Figure 12: Vertical response curve over 3 m intervals
The deviations over 3 m when rotary drilling after a prior 3 m interval of slide drilling were analysed to determine if the previous slide drilling deflection affected the rotary drilling deflection. No apparent relationship was evident. The same was done for rotary drilling versus depth but again no relationship was evident.

The vertical and lateral deviations over each 3 m interval were plotted for both slide and rotary drilling (Figures 14 and 15). The rotary drilling did not create straight boreholes but the deviations were reduced as seen by the tighter grouping in Figure 15.

Although the drillers were limited to rotational speeds below 60 rpm in the pre-drilling instructions, the drillers experimented with traditional rotary drilling variations of increased rotational speed (to 180 rpm) and reduced drilling rate to curve the borehole downwards. Conversely, they reduced rotational speed and increased drilling rate to curve the borehole upwards. The variations in drilling rate are evident in
Figure 10. This was used successfully to extend all boreholes past the no slide depth. The most effective was in borehole DH08 with 408 m being rotary drilled to the final record depth of 2151 m (Table 1).

The drillers also noticed that they were able to manage some semblance of lateral control with the vertical control parameters. Although not recorded until the latest unprocessed drilling, they believed climbing parameters also deflected the borehole to the right and dropping parameters curved the borehole to the left.

The most recent borehole (DH11) has been the most successful with depth drilled before slide drilling could not continue. This borehole is also the borehole with the greatest off-set angle to the target azimuth and has the largest lateral deviation of 166 m to the left. The drilling data has not yet been analysed for this borehole but interest will be in the percentage of rotary drilling used and at what depths in the borehole. This is an indication of improvements in the drillers’ skills through exposure and experience in the drilling practice and innovation on the drillers’ part. This borehole was drilled with a track mounted Series 1000 with higher thrust capacity.

Although the thrust loading is only at approximately 25 % capacity, the rotation pump pressure is at approximately 85 % capacity at depths beyond 2000 m. This is likely to be a limiting factor in determining maximum depth capacity with the current equipment.

The DHM completed six boreholes before being replaced due to bearing pack failure. Wear at the bend is always a problem with DHM drilling but the presence of the 1 mm thick wear pad prevented adverse wear. Some erosion was evident but had not started to penetrate through to the thread at that joint. The PCD drill bit with the repositioned outer cutters that were used over that period showed limited abrasive wear and some chipping on most cutters but not enough to affect the cutting characteristics of the bit.

The survey instrument signal strength reduced rapidly over the first 400 m but from 1200 m, the signal strength remained reasonably constant at 0.5% (Figure 16). The first borehole (EX03) was terminated with survey signal problems. After that, survey signal problems were insignificant with some difficulties only experienced at depths beyond 2000 m with several attempts required occasionally before a signal was received.

With the discontinued supply of the thread grease “Talcor Blue” which had been the standard grease in the industry, the non-metallic grease ERTG 9507 had been recommended as a replacement. For this project, DRTG ZN50 grease with 50% metallic zinc within its formulation was introduced. The grease had good adhesion and anti-galling characteristics and with the metallic content, may have contributed to the improved signal transmission.

The drill rods were comfortably pulled from each borehole without the need of rotation to reduce friction.
CONCLUSIONS

Drilling with a combination of slide and rotary drilling was successful in producing a series of boreholes to and beyond 2000 m and located in positions which should provide adequate gas drainage of the gate-roads before development mining commences.

All components of suggested drilling practice were employed to achieve a successful outcome to the project:

- A larger diameter bit.
- Reduce the effective bend.
- Increase tool-face intervals.
- Rotary/slide drilling.

The straighter drilling provided in the rotary drilled sections reduced in-hole friction and extended the depth to which slide drilling was possible. The reduction in bend magnitude reduced the surging feed usually experienced with slide drilling in long holes. Thrust capacity to overcome friction became the limiting factor on the slide drilling depth rather than repeated stalling of the DHM through surging.

The increase in the bit (and borehole) diameter from the standard 96.1 mm after the first borehole to 99 mm for the remaining drilling would have reduced in-hole friction. The increase in diameter also allowed the DHM with its bend and wear pad configuration to rotate in the borehole without flexing the DHM. This would have reduced friction wear on the bend and reduced the potential of early failure.

Rotary drilling dramatically reduces the thrust friction on rods sliding in a borehole. When slide drilling capacity is reached, rotary drilling can effectively continue the drilling with some lateral and vertical control. Then an intersection with seam roof or floor will terminate the borehole if deviating too far off-line does not terminate the drilling beforehand.

Drill skills and experience in the use of rotary/slide drilling developed during the duration of the project. Rotation capacity and water pump pressure capacity will be the limiting factors for drilling depth capacity when in rotary drilling mode.

Good profile definition from most boreholes provided the mine with definitive RL information over the area of drilling. In the add-on value of exploration provided by in-seam directional drilling, the areas of the longwall gate-roads were shown to be free of structures, which may have adversely affected mining operations. No adverse drilling conditions were experienced which might adversely affect gas drainage from the area.
REFERENCES


DEVELOPMENT OF A PROTOTYPE KEY PERFORMANCE INDICATOR IN LARGE-SCALE DRILLING OPERATIONS

Rohin Simpson¹, Duncan Chalmers² and Farshad Rashidi Nejad³

ABSTRACT: The significant increase of rotary blasthole drilling technology in recent years requires enhanced utilisation tactics of the “Big Data” being produced, to gain insight into operator performance and increase drilling efficiency. An area often overlooked by production is the drill cycle, due to the delayed nature of the downstream effects resulting from poor drilling. The project objective was to develop a prototype Key Performance Indicator (KPI) scorecard model to assess and rank drill operator performance, and provide a two-way feedback mechanism for training and development. Furthermore, to identify applications of the KPI scorecard model to enhance utilisation of Big Data in large-scale drilling operations. The prototype scorecard model was based on three main KPIs - rate of penetration, accuracy to plan, and cycle time. The dataset used to develop the scorecard was based on four different through-seam drill patterns. A main component of the scorecard model is a ranking system that enables feedback on an operator’s proficiency based on these indicators and supporting parameters. By analysing the established scorecard model and ranking system it was found that 13 operators, of the 37 in the dataset, had sufficient data to produce a scorecard and rank them. The results reflect trends in the raw data and provided a strong indication of operator proficiency, identifying areas for improvement.

INTRODUCTION

Many mining operations currently face increasing challenges due to more complex deposits, intensified environmental restrictions and fluctuating commodity prices. These challenges, which will be intensified in the future, urge mining operators to increase efficiency of the operations in order to cut costs. Because drilling has a significant effect on unit operations, i.e. blasting, loading, haulage, crushing and processing, improving the drilling performance is considered a key factor in increasing mining operational efficiency (Hustrulid et al., 2013).

At present there has been a shift in the hard rock mining industry towards a “Mine-to-Mill” or “Drill-to-Mill” optimisation approach. This encompasses a focus on each process, not as separate operations (e.g. drill and blast, production, or the processing plant) but the system as one combined unit and making changes within the system that influence downstream processes, in order to improve the system as a whole (Vynne 2001). By improving drill performance from the base level (i.e. the operator), the flow on effect will be noticed in the downstream process, resulting in productivity improvements across the entire system (Liu and Karen 2001).

As autonomous drilling technologies emerge in the mining industry there is a need for improved operating tactics of drills to deliver increased productivity, drill hole quality and accuracy, improve operator safety, and gaining the maximum benefit from the large amounts of data produced by the drills - often referred to as “Big Data” Knights and Liang (2011). It is therefore essential to utilise these large datasets to gain the maximum benefit through improved identification of root causes, training and performance development and monitoring to improve drilling performance.

PROJECT

This research paper describes the development of a prototype KPI scorecard model to gain more value from “Big Data” provided by the drills. It includes findings of main KPIs, an appropriate and

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comprehensive scorecard layout, the development of a meaningful scoring system to provide unbiased performance rankings, and investigates the application of this scorecard to assess operator performance on automated drills.

Background
The project was conducted at an open cast coal mine in North West New South Wales, Australia. The multi-pit operation uses a dragline, truck and shovel operation, assisted by drilling and blasting, or dozers for rock breakage of thin partings and coal seams. The company is currently seeking a tool to better utilise the “Big Data” produced by their drills to evaluate drilling performance. “Big Data” is a broad term for large complex datasets that present new challenges for traditional data processing methods. Advancements in machine monitoring technology have resulted in vast datasets being produced, yet they are too often left underutilised. The large datasets produced by the drill monitoring system provide potential opportunities to gain more value and operational insight, but require extensive manipulation and processing to produce a useable format from which these benefits can be drawn. A scorecard identifying Key Performance Indicators (KPI) was proposed as a potential method to evaluate and rank operator performance. Presently a drill operator’s performance is largely based on metres drilled per shift or rate of penetration per operating hour. This is decidedly incomplete due to:

- Varying drill pattern layouts and depths;
- Inconsistent geological domains, and rock strata;
- Different models of drill and drill bits used; and
- Whether or not the operator is entering the correct delays into the system.

Prior to carrying out the research the company’s measure for drill performance were directed primarily toward the drill rig and not the individual operator. As such this meant that limited information relating to optimum operating ranges or performance targets were available from the company besides the somewhat crude measures of metres drilled per shift and rate of penetration per operating hour. Consequently, acceptable operating ranges would need to be derived from historical data in order to provide an initial baseline for the prototype scorecard model.

The majority of studies investigating drill performance focus heavily on drill parameters and often overlook the influence of the operator on performance. Liu and Karen (2001), Anon (2010) and Patnayak and Tannant (2005), assist in re-emphasising the importance of the operator on drill performance. A method of comparing operator performance is required to identify a range of operator specific KPIs, bring them into one place, and provide a two-way feedback mechanism for training and development to occur. The ‘Balanced Scorecard’ (BSC) developed by Kaplan and Norton (1992), is a method of tracking and managing company performance to ensure alignment with key targets and long-term strategy. As the BSC approach is more focused on strategic management, a variation of this approach was employed to assess performance from a tactical perspective. Although BSCs do not appear to be widespread in mining, Richard (2004) identifies significant value in adopting this approach for production monitoring and development at an operational level.

The approach taken to introduce a scorecard into the work place should be treated carefully to ensure a positive attitude towards its use is created. This requires the scorecard to maintain a fair and unbiased scoring system and allow active involvement in the development of measures from both management and the operator’s perspectives. Subsequently, the method of ranking operators based on their respective performance scores is intended as a measure to assist the manager with a performance review and used to judge where a particular operator sits in relation to their peers. Although performance rankings may foster competitiveness amongst the operators to improve, it has the potential to be viewed in a negative manner and should be used by managers at their own discretion.
METHODOLOGY

Investigation of operator KPIs
Discussions with experienced drill operators, drill and blast engineers, drill supervisors and management were undertaken to cross-reference a diverse range of viewpoints with the findings in the literature. A range of questions were prepared with the key focus to ascertain the following:

1. Current KPIs
2. Maintenance measures
3. Technical factors
4. Extrinsic/Environmental factors
5. Challenges
6. Variables

It was then necessary to identify targets, upper and lower control limits, and measures of importance set by the company and likewise accepted in industry. A review of the various databases was carried out to determine the available data and its specific format. This included a variety of MS Excel workbooks used to extract data from each of the databases. This data is primarily received from the Aquila Drill System (ADS) installed on the drill rigs. Based on the investigation and comparisons between the literature, experienced mine site personnel and available data, the following variables were chosen to develop the KPI scorecard. It should be noted that these chosen indicators may vary from site to site and are related to the company’s targets for drilling performance and quality.

Chosen KPIs (influenced by the operator)
Dependent variables:
   a. Instantaneous Rate of Penetration (ROP) (m/dr.hr);
   b. Cycle time (min/hole drilled) (this measure is not inclusive of drilling time or operating delays); and
   c. Accuracy to design (m) (this includes two separate measures for the drill hole collar and toe accuracy to the drill pattern design in easting and northing, x- and y-planes respectively).

Supporting parameters (influenced by the operator)
Independent variables:
   a. Weight On Bit (WOB) (kN); rotary speed (RPM); torque (TRQ) (kNm); Air Pressure (AP) (Pa);
   b. Difference in bearing from design (angle˚), difference in mast angle from design (angle˚);
   c. Shift change (hr), lunch break (hr), water or fuel – refilling (hr), fatigue break (hr), machine checks (hr), and accident damage events and duration (hr).

Dependent variables:
   a. Operating time (OT), operating delay (OD), operating standby (OS), no scheduled production (NSP), scheduled loss (SL), unscheduled loss failure (ULF), unscheduled loss other (ULO) - all measured in (hr); availability (% calendar time), utilisation (% of available time), and use of availability (% of available time).

Experimental controlled variables were implemented in the data selection process to reduce variability and minimise errors in the development and analysis of the prototype scorecard model. These are listed as follows:

Controlled variables:
a. Geological domain;
b. Drill pattern design;
c. Drill rig; and
d. Drill bit.

Furthermore, parameters that were seen to be of importance but were either not captured in the data, or found to be unsuitable for application in the scorecard:

Excluded Parameters:

- Drill bit failure modes;
- MTBF (mean time between failure); and
- MTTR (mean time to repair).

These parameters were excluded as they cannot be associated with an individual operator and have been influenced by multiple operators using the same drill. Therefore, cannot be used as objective measures of an operator’s performance in these areas.

Database review

It was apparent that the data stored in each database is setup to assess the performance of individual drill rigs and the drill fleet, which had been done to fulfil company reporting measures as opposed to an operator focused approach. This meant that a significant amount of manipulation was required to extract, filter and organise the data into a usable format for the prototype scorecard development.

Aquila (ADB)

The Aquila Database (ADB) was not directly accessed for the purposes of this project, but it delivers the vast majority of drill data utilised in the production databases. The on-board ADS user interface is used by the operator (Figure 1) to align the drill to the design holes via the screen, align the mast angle, drill heading, drill level in relation to varying floor grades, and a variety of other controls (Caterpillar 2005).

![Figure 1: ADS operator interface (Bucyrus 2011)](image-url)
Production databases

The production databases are primarily used for specific drill information (e.g. hole ID, hole depth, number of holes drilled, drilling time, etc.) and all the time metrics (e.g. OT, OD, OS, NSP, SP, delay reasons and duration).

A range of Structure Query Language (SQL) queries were coded to select and filter data for the specified drill patterns, dates, equipment ID, and to remove various outliers in the data where possible. Testing and validation was required to ensure the SQL queries returned the correct data and to remove errors in the coding.

Performance database

The performance database is setup for the drill fleet and contains a large amount of performance data used in the scorecard development and analysis. The KPI – instantaneous ROP, and supporting parameters – RPM, WOB, TRQ, and AP are recorded at small time intervals in the database for each drill hole and provide a high accuracy of measurement.

Development of KPI scorecard

As no prior scorecard model or similar tool was available from the company or could be found in the literature, a prototype scorecard model had to be developed. This made it challenging to define objective operating ranges for many of the parameters, as limited company targets were available. Subsequently, historical data was required in order to determine a baseline from which to judge performance. Due to this some of the measures may not be entirely objective, although were suitable for the purposes of developing a prototype model to prove the potential benefits of the approach and the dataset was of a significant sample size to justify statistical methods.

The data records used for the scorecard development are from four through-seam drill patterns. Each pattern consisted of over 1000 drill holes with the same burden (m) and spacing (m). Drill holes were angled with typically 15 degrees, with an average depth of 30-50 m; and located in the same geological domain. The three drill-rigs in the dataset were all Bucyrus SKSS model rotary blast-hole drill-rigs, equipped with 270 mm diameter rotary roller tricone drill bits.

A basic layout for the prototype scorecard model was created, shown in Figure 2. The colour code displayed in the diagram indicates the main ‘KPIs’: (i.e. ‘Accuracy’, ‘Cycle Time’, and ‘ROP’) - green, ‘Reference parameters’ - dark blue, ‘Ranking’ - purple, ‘Additional information’ - yellow (e.g. ‘Time metrics’), and ‘Qualitative feedback’ - light blue.

Rate of penetration (ROP)

ROP was chosen as one of the KPIs due to its significance in the literature and as a key company measure in assessing operator productivity. The main ROP units are typically metres per operating hour (m/op.hr). Because ROP per operating hour is influenced by variables such as hole depth, the proficiency of an operator to enter the correct delays and cycle time between drill holes, instantaneous ROP (m/dr.hr) was chosen as the first KPI measure. In doing so a more reliable measure for ROP can be determined and the cycle time (i.e. time taken from when the operator has finished drilling, tramming to the next hole and setting up) can be assessed individually.

Cycle time

Cycle time was previously not recorded by the company but was found to be an important KPI following the research and review with experienced mining personnel. Measuring the cycle time would allow a manager to assess the efficiency of an operator moving between drill holes, setting up the drill and identify any trends in operator behaviour that require further investigation. The defined cycle time per hole does not include the actual drilling time, since this is highly dependent on the depth of hole and ROP. Instead, it was decided to analyse the time taken for jacking, tramming, repositioning and setting up to start drilling the next drill hole. This provides a good indication of the operator’s productivity between drill holes and was deduced from the available data. Equation 1 shows how the cycle time is calculated.
Cycle Time \[ \text{cycle time per hole} \ (\text{min}) = \frac{(\text{Dur} - \text{DT})}{60} \text{holes drilled in period} \]  

where, Dur = operating time (sec) and DT = drilling time (sec).

This KPI is a useful measure to compare against drill-hole accuracy, to determine the balance between accuracy and productivity. This can become a useful comparison if an operator spends too much time lining up the drill to achieve an accurate hole at the expense of productivity, and vice versa.

**Figure 2: Prototype KPI Scorecard model layout**

**Accuracy**

The final KPI was chosen to be accuracy to plan. An inaccurately drilled pattern can lead to poor blast fragmentation, which has a significant impact on downstream processes such as loading and haulage cycles. In order to achieve effective fragmentation and promote an efficient production cycle, it is imperative to produce accurate drilling.

The accuracy is measured in 2-dimensions (2D) by comparing the actual x-coordinate (easting) and y-coordinate (northing) of the collar and toe to design coordinates. The z-coordinates (RL – reduced level) were intentionally omitted due to the unreliable nature of the z-coordinates in the model’s coal seam and bench levels when compared to the actual seam and bench levels. By doing so the variability in measurements are significantly reduced.

The term ‘collar’ refers to the x-, y-, and z-coordinate where the drill hole is started, and the ‘toe’ refers to the x-, y- and z-coordinate where the drill hole finishes. To calculate the specific coordinates, geometry and trigonometric functions were used to ensure an accurate projection was made to the design collar coordinates, as they were previously determined incorrectly in the database. Equation 2 and Equation 3 show how the design collar x- and y-coordinates are calculated, respectively.
\[
CD_X = TD_X + |CA_Z - TD_Z| \times \tan(DM) \times \sin(DH) \tag{2}
\]
\[
CD_Y = TD_Y + |CA_Z - TD_Z| \times \tan(DM) \times \cos(DH) \tag{3}
\]

where, \(TD_X\) = Toe X Design (m) – Easting, \(TD_Y\) = Toe Y Design (m) – Northing, \(TD_Z\) = Toe Z Design (m) – RL, \(CD_X\) = Collar X Design (m) – Easting, \(CD_Y\) = Collar Y Design (m) – Northing, \(CA_Z\) = Collar Z Actual (m) – RL, DH = Design heading (degrees° - from toe to collar) and \(DM\) = Design Mast Angle (degrees° - angle from vertical).

Similar difficulties were encountered with the 2D difference measurement at the toe found in the database. Consequently, the actual toe x- and y-coordinates were projected to the designed z-coordinate (depth) to eliminate inaccuracy created by the difference in z-coordinates between actual and design. Equation 4 is the vector equation for 3D coordinates where, subscripts TD = design toe, TA = actual toe and \(R\) = resultant vector (toe to collar).

\[
\begin{pmatrix}
X_{TD} \\
Y_{TD} \\
Z_{TD}
\end{pmatrix} = \begin{pmatrix}
X_{TA} \\
Y_{TA} \\
Z_{TA}
\end{pmatrix} + t \begin{pmatrix}
X_R \\
Y_R \\
Z_R
\end{pmatrix} \tag{4}
\]

Therefore the actual toe x- and y-coordinates may be determined at the design depth using \(t\) from Equation 4; the calculations are shown in Equation 5 and Equation 6 respectively.

\[
X_{TA@D} = X_{TA} + t \times X_R \tag{5}
\]
\[
Y_{TA@D} = Y_{TA} + t \times Y_R \tag{6}
\]

**Development of a ranking system**

A ranking system was developed to indicate the proficiency of an operator across the main KPIs. This is done by translating drill data into a tangible score and then ranking the respective scores. The rank identifies an operator’s relative performance and allows comparison with the supporting parameters on the scorecard to highlight where improvements can be made. It is intended to assist the manager with the performance review and can be used to monitor the progress of training and development. An individual score is assigned to each KPI, as an operator may excel in one area but need to improve in another. The best total score achievable is 100 points, whereas the worst-case score is zero points. The KPIs were weighted based on their meaning and importance in order to maintain a balance between company targets, the literature, and interview findings.

The weighting for ROP, cycle time and accuracy are somewhat arbitrary but reasonable and reflect the findings and recommendations from management at the particular site for the purposes of developing the prototype scorecard. These are by no means fixed but provide a model to describe how weightings can be assigned to each chosen KPI. Sensitivity analysis was carried out to identify the influence of adjusting these weightings. It was found that the ranking scores do change when the weightings are changed, although they can be easily adapted to suit the preferences of a specific site or at any time during trial implementation of the scorecard. Figure 3 shows the weighting associated with each KPI selected for the ranking system.

The scorecard KPIs, ROP (m/dr.hr) and Cycle Time (min/hole), are typically measured as a combined unit of ROP (m/op.hr), inclusive of cycle time, ROP (m/dr.hr) and OD. Therefore, it was decided to separate ROP (m/dr.hr) and cycle time (min/hole) to better identify trends and root causes.

The accuracy and cycle time scoring ranges aim to achieve the minimal value possible. The scoring system to determine the ranks was ascertained through an iterative process to identify upper and lower control limits, while maintaining a suitable spread that correlated with trends in the raw data and company targets.
Rate of penetration scoring

All ROP data was reviewed and as company targets were previously based on metres drilled per shift there was no baseline or defined range with which to compare the instantaneous ROP data. Subsequently, a baseline was developed using the data provided from the four through-seam drill patterns. The average and Standard Deviation (SD) of the ROP was determined for all the operators and was used as a standard from which to develop the scoring system. There were two components of the ROP score:

- A score based on the difference from the baseline ROP average (representing a target ROP), and
- A score based on the difference from the baseline ROP standard deviation (representing the target range).

The two scores were evenly weighted across the score of 35 with the overall average and standard deviation of the dataset representing a mid-range score.

Using the standard deviation provided a good indication of the variation in the observed ROP when an operator is drilling. Based on the recommendations from site personnel a wider spread of ROP is ideal, as the operator must adjust the WOB and RPM to account for variations in the rock strata and to ensure minimal occurrence of drill bit wear and failure. These supporting parameters directly influence the ROP and this was cross-referenced between the data of known experienced drillers versus an inexperienced trainee driller. Once again this measure is not entirely objective as it is based on the average and standard deviation of historical data, but served as a reference range. As the scorecard is a prototype to convey the benefits that such a model can provide, this was accepted as a suitable approach. It must be noted that during a trial implementation of the KPI scorecard model these baseline target values and optimal operating ranges can be determined to provide objective values from which to base the scoring.

K-factor adjustment for the rate of penetration KPI

Particular adjustments were required for the ROP KPI, as excessive WOB, RPM, TRQ, and AP can have detrimental effects on the drill rig in terms of damage. Therefore, a score system for the reduction of the ROP was developed using the Cramér-von Mises (CVM) test.

The CVM test is a statistical test, also known as the ‘Goodness-of-fit’ test, and is used to find the similarity between two continuous distribution curves shown in Figure 4 (EOM 2011). If the critical value ($w^2$) is greater than the critical value at a particular significance level alpha ($\alpha$) the null hypothesis is rejected. The null hypothesis is that the curves are from the same distribution with a certainty defined by the significance level. Therefore if an operator was operating well outside the normal range a factor can be derived from the critical value.
The theoretical Cramér-von Mises’ integral equation is approximated by Equation 7 and applied to the empirical dataset to determine the Goodness-of-fit (EOM, 2011).

$$w_i^2 \approx \sum_{i=0}^{n-1} (F_n(x_{i+1}) - F^*(x_{i+1}))^2 \times [F^*(x_{i+1}) - F^*(x_i)]$$  \hspace{1cm} (7)

$$0 \leq w_i^2 \leq 1$$

where, $F_n(x)$ = empirical distribution function (i.e. average of all operators); $F^*(x)$ = cumulative distribution function (i.e. operator function), $w_i^2$ – critical value (CVM test statistic).

This was used to determine the difference from the average that an operator performed for the specified operational variables RPM, WOB, AP and TRQ, known as the critical value ($w_i^2$). The smaller the critical value determined, the less similarity between the curves. Following the determination of these factors, a K-factor ($K$) was determined and is shown in Equation 8.

$$K = \Upsilon_{RPM} \times \Upsilon_{AP} \times \Upsilon_{TRQ} \times \Upsilon_{WOB} \leq 1$$  \hspace{1cm} (8)

where, $\Upsilon_{RPM}, \Upsilon_{AP}, \Upsilon_{TRQ}, \Upsilon_{WOB}$, are adjustment factors for RPM, AP, WOB, TRQ respectively derived from $w_i^2$.

The K-factor was used to reduce the operator’s ROP ranking score if their ROP was deemed significant by CVM’s test. The adjustment factor may be used in the scorecard with varied significance levels as desired by the user.

**RESULTS**

**Operator rankings and KPI scorecard model**

The results of applying the scoring system to the 13 operators are shown in Table 1, with the score and associated rank achieved in each category. The total ranks show that an operator does not necessarily have to be amongst the best operators in each individual category to achieve a high total rank. This can be seen in the rankings of the three highest ranked operators. For example Operator A ranks first overall, and only ranked fourth in the accuracy; also Operator M ranked second overall, yet only ranks seventh in the ROP category.

The total scores of the operators are well spread. One operator achieved a ranking of 79 points, indicating excellent drill performance based on the developed scoring system. The prototype scoring scale shown in Table 2 could be used to rate the operator overall performance scores.
Most operators scored average (50 points) or above average. Only four operators have a score below the average and two out of these four operators are very close to the average performance score of 50; namely Operator K and Operator F at 49 and 48 points respectively. Only Operator H and Operator C have a score well below average. The results of the rankings look reasonable and could be attributed to the large dataset that was analysed. This is a good sign in terms of reducing potential errors. But it has to be considered that this system is only a prototype model, which requires implementation and an iterative development process of refinement. The total ranking shows that the system balances out potential bias and assesses performance based on multiple weighted parameters. As an example, Operator B was selected to discuss the prototype KPI scorecard and how it may be used for a performance review in detail with reference to the associated ranking. Operator B ranks highly (1st rank) in regards to accuracy, although there shows poor performance in terms of cycle time (11th rank).

### Table 1: Final rankings summary of results

<table>
<thead>
<tr>
<th>Operator</th>
<th>Accuracy</th>
<th>ROP</th>
<th>Cycle Time</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Score</td>
<td>Rank</td>
<td>Score</td>
<td>Rank</td>
</tr>
<tr>
<td>Operator A</td>
<td>28</td>
<td>4</td>
<td>27</td>
<td>2</td>
</tr>
<tr>
<td>Operator M</td>
<td>32</td>
<td>1</td>
<td>17</td>
<td>7</td>
</tr>
<tr>
<td>Operator E</td>
<td>26</td>
<td>5</td>
<td>24</td>
<td>4</td>
</tr>
<tr>
<td>Operator G</td>
<td>31</td>
<td>3</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Operator I</td>
<td>16</td>
<td>12</td>
<td>28</td>
<td>1</td>
</tr>
<tr>
<td>Operator D</td>
<td>24</td>
<td>7</td>
<td>16</td>
<td>9</td>
</tr>
<tr>
<td>Operator L</td>
<td>25</td>
<td>6</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>Operator B</td>
<td>32</td>
<td>1</td>
<td>18</td>
<td>6</td>
</tr>
<tr>
<td>Operator J</td>
<td>23</td>
<td>8</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>Operator K</td>
<td>21</td>
<td>9</td>
<td>17</td>
<td>7</td>
</tr>
<tr>
<td>Operator F</td>
<td>20</td>
<td>11</td>
<td>26</td>
<td>3</td>
</tr>
<tr>
<td>Operator H</td>
<td>21</td>
<td>9</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>Operator C</td>
<td>12</td>
<td>13</td>
<td>9</td>
<td>13</td>
</tr>
</tbody>
</table>

**Review of rankings against KPI scorecard model**

Operator B’s scorecard is based on 15 shifts within the period from 24th of January to the 19th of August 2014, three Bucyrus SKSS model drill rigs were operated during the drilling of four associated through-seam drill patterns. Operator B drilled 175 holes in this period with an average depth of 40.14 m. A total 7,025 m were drilled during the 15 shifts, which equates to about 470 m/shift on average (Figure 6). Referring to the company goals, this is just a bit under the target of 500 m/shift.

The graphs ‘Collar Accuracy’ and ‘Toe Accuracy’ indicate average 2D difference of the actual collar and toe positions compared to design. In both graphs it can be seen that Operator B is more accurate than the average. Furthermore, the standard deviation of Operator B is relatively small compared to the overall standard deviation. A similar trend can also be seen in the ‘Bearing’ and ‘Mast Angle’ charts. The accurate adjustment of the bearing and mast angle support the high accuracy results of the toe location and is reflected in the rankings.

Operator B ranks sixth for the ROP ranking. In Figure 6 the cumulative frequency plots for RPM, WOB, TRQ and AP, show Operator B very close to the average. Although the ROP of Operator B appears to be a bit below the average a mid-rank 6th out of 13 (seen in Figure 6) is a reasonable result.

By looking at the graph ‘Cycle time per hole’ (Figure 6), it is found that Operator B’s average cycle time per hole exceeds the average cycle time of the dataset. The numbers reveal that Operator B requires around 30% more time to drill a hole compared to the average. These numbers may indicate that Operator B spends too long tramming and lining up the drill, decreasing productivity. The utilisation of available time is slightly below the average, which could be attributed to the poor cycle time performance. Operator B is ranked 11th in the Cycle Time category (Figure 5). Based on the results it is observed that
Operator B’s cycle time is an area for improvement. The poor cycle time performance could be connected to excessive attention to achieve excellent drilling accuracy. One approach could be to analyse the operating delays and standbys (seen in Figure 5) in order to prove if Operator B had been recording delays properly (e.g. lunch break which appears inconsistent on the charts, or other delays not listed). Any feedback from Operator B could be helpful to identify reasons for the cycle time performance in order to provide training and development, if required.

### Table 2: Operator proficiency scores for ranking system

<table>
<thead>
<tr>
<th>Total Score</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>81–100</td>
<td>Outstanding</td>
</tr>
<tr>
<td>71–80</td>
<td>Excellent</td>
</tr>
<tr>
<td>61–70</td>
<td>Very Good</td>
</tr>
<tr>
<td>46–60</td>
<td>Average</td>
</tr>
<tr>
<td>35–45</td>
<td>Poor</td>
</tr>
<tr>
<td>&lt;35</td>
<td>Very Poor</td>
</tr>
</tbody>
</table>

**Figure 5: Operator B’s prototype KPI scorecard** (page 2)

**Figure 6: Operator B’s prototype KPI scorecard**

**ANALYSIS**

### Scorecard and rankings

The KPI scorecard was able to identify operator performance and substantiate some potential reasons for each KPI ranking based on the supporting parameters. This is in contrast to the majority of literature, which appears to relate successful operating performance primarily on ROP and associated drill parameters such as WOB and RPM. It was found that a consistent performance across the board is the main key in gaining a high total score.

The analysis of the operator rankings based on the developed scoring system show clearly that operators have strengths and weaknesses in the various areas of performance. This verifies the need for a
balanced scoring system to assess the performance based on multiple parameters. By comparing the operator ranks across the different KPIs it was found that the ranks varied between ROP, accuracy and cycle time. This finding supports the proposed assumptions and methodology that a scorecard can provide a more encompassing view on operator performance, and that the ranking system can eliminate bias towards a specific KPI measures, such as ROP. The detailed example review of Operator B’s scorecard revealed that the prototype KPI scorecard model is a strong tool to identify areas for improvement and development, which may lead to enhanced productivity.

The scorecard approach brings the large amounts of data, referred to as “Big Data”, into one place where a supervisor, manager and also operator can look at various aspects of the operator’s performance. It also incorporates company targets and strategy in order to align and compare an operator’s performance in relation to these measures and the other operators. This is further supported by the views of Knights and Liang (2011) who mention that effective utilisation of production data can provide a key insight into a mining operation if managed correctly. Furthermore, the KPI scorecard model provides a mechanism to allow a two-way feedback process to occur between manager or supervisor and the operator. It should be used as a constructive tool and not considered as a negative thing by the operator. This is important to ensure that successful implementation is achieved. Moorraj et. al (1999) highlight the importance of a constructive learning environment based around the use of a performance scorecard. The scorecard is also an important tool to enhance key themes such as communication and engagement between both leader and operator, which are often found to be lacking in employer-employee relationships. Increased levels of engagement can lead to increased productivity in most cases.

CONCLUSIONS

The purpose of developing a prototype KPI scorecard model was to demonstrate the potential applications to enhance drilling performance influenced by the operator, improve the feedback and development process, and thereby decrease the impact of poor drilling on blasting, loading haulage and even downstream processes. By reviewing and comparing all available resources, both qualitative and quantitative, it was found that the main KPIs for the development of the prototype scorecard are accuracy (i.e. 2D collar and 2D toe accuracy), adjusted ROP (considering RPM, WOB TRQ, and AP), and cycle time (operating time excluding drilling time and delays).

The established scorecard shows the KPIs in a graphical representation, contains general information, and provides reference parameters that are used to draw conclusions based on the findings from the indicators. This was found to be very useful as it provided an initial indication of the operator’s strengths and weaknesses in these areas. The ranking is intended as a measure to give an initial indication of the performance in each KPI. The reviewer should still use their judgement and apply discretion based on experience when providing operator feedback. Clear communication and engagement is essential to ensure successful implementation. It is also important that operator feedback regarding the measures on the scorecard, and any areas where the model could be improved, are taken into consideration to maintain the iterative development of the KPI scorecard model.

RECOMMENDATIONS

During the process of developing the prototype scorecard, various areas for potential improvement were identified. The developed scorecard is only a prototype and was not trialled in operation. In order to validate the benefit of the scorecard in terms of improving the performance based on feedback provided from the scorecard, it is recommended to introduce trial scorecards in the field. The prototype scorecard model can be tailored to site and company specific targets, and specified operating ranges as these vary between different mining operations and between different drill rig models and specifications. Feedback from the operators and insight from manager and supervisors should then be used to adapt and improve the scorecard model where required. It is also recommended to investigate whether the ranking system weighting is appropriate. This is highly dependent on company targets and may change with time. The KPI scorecard layout could be slightly adapted to show only the raw score and the manager could look at
the rankings separately from the scorecard. It could be beneficial to implement a measure for drill bit consumption and failure modes attached to an operator's name and relate this to the ROP measure and supporting parameters. This would provide an additional method to relate the operator's influence on the supporting parameters WOB, RPM, TRQ and AP to excessive drill bit wear and failure mechanisms, to establish acceptable operating ranges and incorporate these into the ROP scoring system. A financial measure could also be included to give an indication of an operator's cost per drilled metre to allow management to gauge the costs associated with each operators drilling proficiency.

Finally, it is recommended that the KPI scorecard model has the potential for application across any mine site that has rotary drills. This would require minor changes and development to tailor the scorecard to a specific site or company, but by and large in large the fundamental concept would remain similar. Furthermore, not only could this concept be applied to drilling operations to improve and develop operator performance from a tactical level, but it could also be applied to a vast number of operational roles in mines and other industries, with relevant modifications.

ACKNOWLEDGEMENTS

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REFERENCES


EOM - see Encyclopedia of mathematics.


NUMERICAL SIMULATION OF INTEGRATED RESERVOIR-BOREHOLE FLOW FOR PRE-MINING DRAINAGE

Mohsen Azadi, Saiied Mostafa Aminossadati and Zhongwei Chen

ABSTRACT: The accumulation of methane in coal seams and surrounding geological structures as well as underground coal mines has been the major contribution to gas outbursts and mine explosions. Drainage of Coal Seam Gas (CSG) prior to mining using Surface to In-seam (SIS) and Underground In-seam (UIS) boreholes is crucial to reducing the potential risk to the safety and productivity of underground mining operations. Many researches have been carried out to identify the factors affecting the gas drainage performance such as coal properties, gas content and drainage borehole geometries. Two different flow conditions determine the gas drainage efficiency: borehole flow with injection from wall and reservoir flow in a porous medium. These two different types of flow have previously been studied separately. However simultaneous flow of gas through reservoir and borehole requires further investigation.

In this research, a three dimensional model for simulation of integrated reservoir-borehole flow is developed to study the significant effect of borehole geometry on flow characteristics of coal seams. Computational Fluid Dynamics (CFD) simulations were carried out using finite volume based software ANSYS Fluent. Four different borehole diameters of 7.5, 10, 12.5 and 15 cm as well as three different lengths of 50, 100, and 150 m were chosen to accomplish the parametric study of borehole geometry. It is assumed that the boreholes are in a steady state condition for two different single phase scenarios of liquid flow (water) and gas flow (methane). The CFD simulations are validated with previous pressure drop models for internal single phase gas and liquid flow. The obtained results reveal that increasing the borehole diameter leads to reduction in fluid pressure throughout the coal seam. On the effect of borehole length it is seen that at a specific distance from borehole outlet, the pressure distribution is independent of the borehole length and upstream effects.

INTRODUCTION

Many engineering and industrial applications still rely on coal as a major energy source. Coal seam reserves contain a considerable amount of gas. In a general estimation, the gas content for different types of coal varies between 0.1 to 25 cubic meter of gas per ton of coal. Coal seam gas (CSG) is mainly composed of methane which is estimated at 80%-95% of overall gas content. Methane gas is removed prior to mining to ensure the safety of mining workings. The challenges involved in coal extraction are growing remarkably as underground mines are becoming deeper, gassier and more complicated in geometry.

Mining pre-drainage is the most important prerequisite for removing methane gas from deep and gassy coal reservoirs to achieve a safe environment for mining exploitation operations. In addition to mining concerns, this process leads to gas production as another valuable source of energy. In spite of significant progress in the development of underground mining technologies and improvement of mine safety, there are still fatal accidents and explosions happening in underground coal mines.

One of the major concerns related to mining pre-drainage is gas ventilation control and management. Two major method are used to satisfy the required safety standards in terms of reservoir gas content: i) Surface to In-seam (SIS); and ii) Underground In-seam (UIS) drilling of boreholes for water and gas drainage. To develop these boreholes, drilling is conducted directionally from vertical to horizontal sections with different diameter ranges for the purpose of gas content reduction from the coal.
A reliable prediction of coalbed methane flow depends on the different mechanisms concerned with coal structure and reservoir properties as well as drainage borehole geometry. Accordingly, many studies have been performed focusing on either reservoir simulations or borehole flow and pressure drop predictions. However, most of these investigations are basically designed for oil and gas applications with more focus on reservoir engineering aspects. In comparison, less attention has been paid to CSG flow studies with specific focus on borehole impacts on simultaneous flow of gas through coal seam and boreholes.

Many studies have been carried out to simulate flow of fluids from different types of reservoirs into wells or boreholes (Jenkins and Aronofsky, 1953; Aronofsky and Jenkins, 1954; Al-Hussainy et al., 1966; Yao et al., 2013). Early theoretical models or numerical simulations were designed for oil and gas applications. Jenkins and Aronofsky (1953) presented a numerical method for describing the transient flow of gases in a radial direction for a porous medium for which the initial and terminal pressure and/or rate are specified. They developed a simple means for predicting the well pressure at any time in the history of a reservoir. In their next study (Aronofsky and Jenkins 1954) suggested an effective drainage radius was for which steady state gas flow assumption could be used to predict well pressure in the process of gas reservoir depletion. In a rigorous model Al-Hussainy et al., (1966), considered the effect of variations of pressure dependent viscosity and gas law deviation factor on the flow of real gases through porous media. They used pseudo-pressure as change of variable to reduce the equations to a form similar to diffusivity equations. Yi et al., (2009) simulated gas flow through a reservoir using two dimensional solid-gas coupled software RPFA to study the effect of permeability, borehole spacing and diameter and gas content on reservoir pressure and drainage radius. Packman et al., (2011) used SimedWin to simulate CSG flow in an attempt to demonstrate the ability of enhanced gas recovery to increase gas flow rate. Based on their reservoir model calibrated by history matching, they concluded that with regard to increased gas flow rate and decreased drainage time, enhanced gas recovery through injection of nitrogen is achievable. Most of these researches have focused only on reservoir aspects of simulation and their assumptions need further investigations in terms of flow dimensions. The errors concerned with simplifying assumptions limit the range of application of these reservoir simulators. Moreover, borehole flow is defined as a boundary condition and is not included in the mathematical modelling and governing equations of the reservoir simulators. These assumptions neglect the interactions at reservoir and borehole interface and need further attention.

On the effect of borehole wall influx or outflux, a number of studies have been carried out to understand the flow filed behaviour and pressured drop along boreholes (Asheim et al., 1992; Yuan 1997; Su and Gudmundsson 1998; Yuan et al., 1999). Siwon (1987) developed a one-dimensional model for steady state flow of incompressible fluid in a horizontal pipe perforated with circular orifices. Ouyang et al., (1998) continued this study by developing a pressure drop model for pipes with perforated wall that can easily be used in reservoir simulators or analytical models. This model considers different types of pressure drops including: frictional, accelerational, gravitational as well as pressure drop caused by inflow. They concluded that for laminar flow, wall friction increases due to inflow whereas for turbulent flow wall friction decreases as a result of inflow.

Based on this approach, more attempts have been carried out to obtain the most accurate pressure drop models for borehole flow. Yalniz and Ozkan (2001) investigated the effect of inflow from horizontal wall on flow characteristics and pressured drop experimentally and theoretically. They developed a generalized friction factor correlation that is a function of Reynolds number, the ratios of inflow to wellbore flow rate and perforations to wellbore diameter. Wang et al., (2011) measured pressure drop due to inflow in a horizontal perforated pipe loop by using water as working fluid. Their experimental results show that pressure drop grows as a result of increased injection flow rate. They developed a model that suggests that total pressure drop consists of two parts including perforated pipe wall friction loss and an additional pressure drop term. In a recent study, Zhang et al., (2014) presented a comprehensive model for prediction of pressure drop based on the previous studies and some new experiments. Their results show that this model presents more accurate results compared to previous models and can also be used for a wider application range. It must be noted that none of the these
studies has been conducted to develop a model for prediction of pressure drop and production rate for coal seam boreholes with inflow and most models developed so far are derived for oil and gas flow conditions.

In addition to theoretical models, some researchers have simulated borehole flow using numerical techniques to avoid the simplifying assumption (Folefac et al., 1991; Seines et al., 1993; Siu et al., 1995; Su and Lee 1995; Yuan et al., 1998; Ouyang and Huang 2005). Guo et al., (2006) developed a numerical model to study the deliverability of multilateral wells. Their model was capable of coupling the inflow performance of the individual laterals with hydraulics in curved and vertical well sections. Zeboudj and Bahi (2010) simulated wellbore flow with pipe injection using Computational Fluid Dynamics (CFD) simulation as a replacement for further experiments. They discussed the experimental measurement shortcoming in the assumption of a constant momentum-correction factor which is not true in the case of wall inflow. CFD simulation, however, allows the exact calculation of this parameter by considering all variations of velocity in radial direction eliminating the need for making flawed assumptions. In another study, Ouyang et al., (2009) studied single-point wall entry for oil and gas wellbores. The significant effect of borehole hydraulics on production predictions, performance evaluations and completion design for horizontal and multilateral boreholes needs to be well understood. In this respect, they used CFD modelling using ANSYS to investigate flow profiles and pressure distribution along the wellbore thoroughly. Their simulation results showed that moving the entry point closer to the outlet section reduces the significant impact of inflow on the total pressure drop along the borehole. The simplifying assumption of constant and pre-defined wall inflow rate needs to be improved and evaluated further.

Depending on borehole geometry the flow characteristics through the coal seam and borehole may vary. Some theoretical models and reservoir simulators have been presented accordingly. However, most of them are either inaccurate due to simplifying assumptions or designed mainly for oil and gas or shale gas reservoirs. This is why operational experience, which is basically subjective, is still considered as an essential requirement for efficient gas drainage of coal seams. Efficient drainage of coal seams prior to mining requires a good understanding of reservoir and borehole conditions and their interactions. In this study, a large scale three dimensional model is developed using CFD simulations to study the integrated reservoir-borehole flow during coal seam drainage. The significant influence of borehole diameter and length on the coal seam flow behaviour is investigated.

**MATHEMATICAL MODELLING**

**Model assumptions**

Coal seams are generated by compression of plant and animal matter over millions of years. During this process CSG is trapped inside the coal seam by water and ground pressure. The methane gas is lied inside the coal matrix sealed with water existing in coal I fractures which are called cleats. As the reservoir pressure at wellbore falls the water begins to move out of cleats letting the gas be desorbed from the coal matrix. Based on the described drainage process, the following assumptions have been taken into consideration:

- Water was considered as working fluid for single phase liquid flow
- Methane as a compressible ideal gas was considered as working fluid for single phase gas flow
- The simulations are conducted in the single phase production phase in steady state condition
- Two cell zone conditions for porous coal seam and internal borehole flow were considered
- Coal is considered as a homogenous porous media holding gas in the coal matrix
- Fluid flow through the fracture network of coal obeys Darcy’s law
- No borehole boundary condition was defined at the borehole wall
- The flow variables are transferred between borehole and porous zone by defining an interface at the contact region of the two zones
- Flow through the borehole is considered turbulent
- Flow through the coal seam is considered laminar
One of the most determining parameters affecting drainage of coal seam is reservoir permeability. Depending on coal seam depth, the reservoir can be classified into three groups of shallow, medium-depth and deep. Coal permeability varies from near 0.1 to 100 \( md \) for deep and shallow reservoirs, respectively (Darling 2011). In this study horizontal and vertical permeabilities of 10 \( md \) and 1 \( md \) were considered for coal seam zone, respectively.

**Governing equations**

Based on the mentioned assumptions two different sets of equations are required to simulate flow through the borehole and coal seam. Flow in the borehole section is considered internal turbulent pipe flow with distributed mass transfer through then wall and flow through coal seam is treated as a porous media.

**Borehole flow equations**

Considering varying mass transfer through borehole wall resulted from reservoir drainage the conservation equations of mass momentum and energy can be written as follows:

\[
\frac{\partial}{\partial t}(\rho u_i) = 0 \tag{1}
\]

\[
\frac{\partial}{\partial x_j}(\rho u_i u_j) = -\frac{\partial P}{\partial x_j} + \frac{\partial}{\partial x_j}\left[\mu \left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} - \frac{2}{3} \delta_{ij} \frac{\partial u_k}{\partial x_k}\right)\right] + \frac{\partial \tau_{ij}}{\partial x_j} + \rho \ddot{g} \tag{2}
\]

\[
\nabla \cdot (\rho \dot{v} (\rho E + P)) = \nabla \cdot \left(k_{eff} \nabla T + (\tau_{ij} \cdot \dot{v})\right) \tag{3}
\]

Where:

\[
\tau_{ij} = -\rho u_i u_j \tag{4}
\]

\[
E = h - \frac{\rho}{2} + \frac{v^2}{2} \tag{5}
\]

In the above equations, \( \tau_{ij} \) is the Reynold stress tensor which represents the effect of turbulent fluctuations on fluid flow. This term was computed using standard \( k - \varepsilon \) turbulence models to close the mass and momentum equations. For the Energy equation, \( k_{eff} \) is the effective conductivity which is equal to \( k + k_t \), where \( k_t \) is the turbulent thermal conductivity, defined according to the turbulence model being used. The second term on the right-hand side of Eq. (3) represents energy transfer due viscous dissipation. The details of turbulence models used in the current study with all the constant values can be found in FLUENT theory guide (2011).

**Reservoir flow equations**

Since the volume blockage that is physically present is not represented in the model, a superficial velocity inside the porous medium was used, based on the volumetric flow rate, to ensure continuity of the velocity vectors across the porous medium interface. The porous media is modelled by the addition of a momentum sink term to the standard fluid flow equations. To do this, Darcy flow is considered through the coal fracture network. Under the suggested assumptions for coal seam zone, the conservation equations are written below:

\[
\frac{\partial}{\partial x_i}(\rho u_i) = S_m \tag{6}
\]
\[
\frac{\partial}{\partial x_j} (\rho_i u_j) = -\frac{\partial P}{\partial x_j} + \frac{\partial}{\partial x_j} \left[ \mu \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} - \frac{2}{3} \delta_{ij} \frac{\partial u_k}{\partial x_k} \right) + \frac{\partial \tau_{ij}}{\partial x_j} + \rho \bar{g} + \bar{S}_i \right]
\]

where \( S_m \) is the mass source term accounting for the desorption of gas from coal matrix and:

\[
\bar{S}_i = -\frac{\mu}{k} \bar{v}_i
\]

This momentum sink contributes to the pressure gradient in the porous cell, creating a pressure drop that is proportional to the fluid velocity in the cell.

ANSYS FLUENT solves the standard energy transport equation (Eq. 3) in porous media regions with modifications to the conduction flux. For simulations in which the porous medium and fluid flow are assumed to be in thermal equilibrium, the conduction flux in the porous medium uses an effective conductivity:

\[
\nabla \cdot \left( \bar{v} (\rho E + P) \right) = S_f^h + \nabla \cdot \left( k_{\text{eff}} \nabla T + \left( \tau_{\text{eff}} \cdot \bar{v} \right) \right)
\]

where \( \rho \) is fluid density, \( \rho_s \) is solid medium density, \( \varphi \) is porosity of medium, \( k_{\text{eff}} \) is effective thermal conductivity of medium and \( S_f^h \) is fluid enthalpy source term.

**Computational model**

A UIS borehole drilled through a section of coal seam is chosen as the base physical model. A 100 × 5 m coal panel with seam thickness of 2.5 m and a borehole of 10 cm in diameter was considered as the baseline condition. User defined mass source term compiled in C language were implemented in Fluent solver to account for desorption of fluid from the porous coal seam zone. Outlet atmospheric pressure boundary condition at the borehole end was assumed. Four different borehole diameters of 7.5, 10, 12.5 and 15 cm as well as three different lengths of 50, 100, and 150 m were chosen to accomplish the parametric study of borehole geometry. The coal seam-borehole models generated for the current simulations are presented in Figure 1.

The Semi-implicit Method Pressure-linked Equations (SIMPLE) algorithm was used for the pressure-velocity coupling. The second-order upwind discretization scheme was utilized for momentum, turbulent kinetic energy, and turbulent dissipation rate. The computations were carried out using parallel processing on a high performance computing workstation with 12 nodes. Each node is configured as follows: 2 × 10 cores @2.60GHz, 128GB RAM.
RESULTS AND DISCUSSION

Validation of the model

From the baseline condition, the borehole diameter and length were varied to accomplish a valid parametric study of integrated coal seam-borehole flow. All the simulations were run for both methane flow and water flow as the working fluids during pre-mining drainage of underground coal seams.

The computed results for methane flow through borehole were compared with Atkinson’s equation (Le Roux 1990) to give the pressure drop using the following equation:

\[ \Delta P = \frac{k_{er} L}{A^3} \frac{\rho}{\rho_{air}} Q^2 \]  

where \( \Delta P \) is the pressure drop (Pa), \( k \) is Atkinson friction factor (kg/m³), \( P_{er} \) is borehole perimeter (m), \( A \) is cross-sectional area (m²), \( \rho \) is gas density (kg/m³), and \( Q \) is gas flow rate (m³/s). The computed pressure drops for four different diameters (coloured with diameters) as well as three different lengths at \( x=50 \) m for borehole diameter of 10 cm are presented in Figure 2. The simulation results show good agreement with Atkinson’s equation. For water flow, the model results were compared with the following pressure drop model along pipes (Aziz and Govier 1972):

\[ \Delta P = \frac{k_{air} L}{A^3} \frac{\rho}{\rho_{air}} Q^2 \]
Figure 3: Comparison of simulated model for water flow with (Aziz and Govier 1972) correlation

\[
\Delta P = 2f \frac{\rho V^2 L}{D}
\]

\[
f = \begin{cases} 
  \frac{16}{Re} & \text{for } Re \leq 2200 \\
  0.077716 \left( \log \left( \frac{6.9}{Re} + \left( \frac{\varepsilon}{3.7D} \right)^{1.11} \right) \right)^{-2} & \text{for } Re > 2200
\end{cases}
\]

where \( Re \) is the Reynold number, \( \varepsilon \) is the absolute pipe roughness. Same geometries as described for methane flow are used this time for water flow (Figure 3). As can be seen, the obtained results show good agreement with the pressure drop model along pipes.

Development of a three dimensional and integrated model through coal seams can be used as a promising tool to improve our understandings about flow field variables and behaviour. The velocity streamlines through coal seam and borehole are illustrated in Figure 4. As presented in this figure, fluid flow originates from coal matrix and is injected to boreholes due to near borehole effects and negative pressure gradient. These results are essential for advancement of borehole development plans and efficient drainage methods where few in situ data are available due to access limitations and geometrical difficulties. Another advantage of the current model is providing flow field data at any point through the coal seam for any given geometry and operating condition using a fast and cost effective computer model.

Effect of borehole diameter

Pressure contours at five planes (\( x=0, 25, 50, 75, 100 \) m) along and three planes (\( z=0, 2.5, 5 \) m) across the coal seam for single phase gas and water flow are illustrated in Figure 5. The obtained results show that by increasing the borehole diameter the fluid pressure throughout coal seam falls resulting in more efficient drainage of the coal seam. This behaviour can be explained by bigger drainage area and
smaller pressure drop along the boreholes and proves the significant influence of borehole flow on pressure distribution through reservoir.

Figure 4: Velocity streamlines through coal seam reservoir and borehole

(a)

(b)
Figure 5: Pressure contours along coal seam for different borehole diameters for: a) methane flow, and b) water flow

To scrutinise the effect of borehole diameter on coal seam pressure distribution closely, the pressure profiles in horizontal and vertical direction across coal seam were plotted at \(x=50\ m\) (Figures 6-7). As expected, moving from borehole to coal seam in both horizontal and vertical direction, the pressure grows sharply until reaching nearly a constant value far from borehole. A close comparison of pressure distributions for methane and water flow reveals that pressure variations under the effect of borehole diameter are more significant for water flow than methane flow.

Figure 6: Pressure distribution in Z direction across coal seam at \(x=50\ m\) for: a) methane flow, and b) water flow
Velocity profiles for four different borehole diameters along the borehole centreline for methane and water flow are presented in Figure 8. As expected, the velocity magnitude varies inversely with borehole diameter to satisfy the continuity of mass flow rate at the borehole outlet for similar fluid production from the coal seam. Velocity profile along a vertical direction at three different sections along borehole (x=1, 50, 100 m) for methane and water flow are presented in Figure 9. It is observed that velocity magnitudes across boreholes are remarkably larger than through porous coal seam. It can also be seen that moving from coal seam end to outlet section, the velocity magnitude increases considerably due to continuous injection of fluid along the borehole.

Effect of borehole length

Pressure contours for different borehole lengths at three planes with similar distance from borehole outlet (L-x=0, 25, 50 m) and three planes (z=0, 2.5, 5 m) across the coal seam for single phase water flow are presented in Figure 10. These three planes along the borehole were chosen to investigate the influence of upstream effects on drainage behaviour and pressure distribution through coal seams with longer boreholes. Pressure through the coal seam in the far from borehole regions does not vary significantly along the coal seam in the x direction. This behaviour can be explained by the greater value of coal permeability in the horizontal plane compared with the vertical plane. The computed
results indicate that for a specific distance from the borehole outlet, the pressure distribution is almost independent of borehole length and upstream effects. This behaviour is investigated further by plotting pressure profiles across the horizontal and vertical directions through coals seams of different lengths ($x=50,100,150$ m) as presented in Figure 11. As can be seen, the curves overlap which confirms the previous interpretations.

Figure 9: Velocity profile along Y direction for methane (left) and water (right) flow at: a,c) $x=1$ m; b,d) $x=50$ m; e,f) $x=100$ m
Figure 10: Pressure contours along coal seam for different borehole lengths

Figure 11: Pressure distribution at distance of 25 m from borehole outlet in: a) Y direction, and b) Z direction

Velocity profiles across the vertical direction at a distance of 25 m from the borehole outlet for three different coal seam lengths \(x=50,100,150\) m, are presented in Figure 12. As one can be seen, the longest coal seam has the highest velocity magnitude across the borehole which can be explained by higher injection from upstream to borehole for longer coal seam case. Same findings presented for Figures 10-12, were observed for the effect of borehole length on single phase methane flow through coal seam and borehole.
CONCLUSIONS

A three dimensional CFD model for simulation of integrated reservoir-borehole flow is developed to study the significant effect of borehole geometry on flow characteristics of coal seams. Four different borehole diameters and three lengths were simulated for single phase methane and water flow. Using computer simulations, it was shown that by increasing the borehole diameter, the fluid pressure throughout the coal seam falls resulting in more efficient drainage of the coal seam. It can also be seen that velocity magnitude is remarkably large across borehole than through porous coal seam and moving from coal seam end to outlet section, the velocity magnitude increases considerably due to continuous injection of fluid along the borehole. A close comparison of pressure distributions for methane and water flow reveals that pressure variations under the effect of borehole diameter are more significant for water flow than methane flow. Pressure through the coal seam in the far from borehole regions does not vary significantly along the coal seam in the x direction. In addition, the computed results indicate that for a specific distance from the borehole outlet, the pressure distribution is almost independent of borehole length and upstream effects. This study proves that the presented CFD model can be used as a promising tool for pre-mining drainage simulations. This model can provide the mining industry with in situ data using inexpensive, flexible and fast computer simulation.

REFERENCES

Darling, P, 2011. SME Mining Engineering Handbook. Littleton, SME.

Figure 12: Velocity profile along Y direction at the distance of 25 m from borehole outlet for different borehole lengths


A FIELD EVALUATION OF A MAIN AXIAL VENTILATION FAN TO ESTABLISH STALL ZONE AND FAN PERFORMANCE CURVE

Tim Harvey¹ and Bharath Belle²

ABSTRACT: This paper summarises the approach taken in establishing and validating the stall zone of a main axial fan for an underground expansion project. The expansion of an operating underground bord and pillar mine required the use of a second fan in parallel at the start of the development project. To improve the level of confidence in the ventilation simulation model that incorporated an existing fan curve provided by the fan supplier necessitated an independent fan test. Therefore, performance and stall characteristics of the current fan and pressure-quantity (PQ) survey of the mine was conducted and the results were used to calibrate the ventilation simulation model. The main axial fan was tested through a range of operating points beyond the currently perceived pressure stall point of 2.1 kPa. A pitot traverse was conducted for two operating points and the remaining operating points were measured on fan instruments. This paper details the test procedures and instruments used to collect and analyse data, and the theory used in analysis and to calibrate fan differential pressure flow measurement instruments. The fan test study has validated the fan curves with different pitch setting for use in ventilation simulation studies with twin fan installations operating with the fan pitch set to 17.5º, which is to give a good operating safety margin from the stall zone of 2.6 kPa.

INTRODUCTION

Main ventilation fan is a key safety and business critical control for underground risk management. One of the key requirement of mine fans is that they are robust, reliable and have the flexibility to provide required pressure and air quantities to the mine design requirements. Unintended fan performance would result in ventilation conditions that may create hazards such as elevated levels of hazardous gases, pollutants, stoppages of working places and evacuation of workers from the underground environment (MDG3 2015). Current or new mine projects seek to utilise the existing mine or spare main fans from an operating mine. This paper summarises the evaluation of one such axial fan used in the pre-feasibility expansion study of a care and maintenance bord and pillar mine. The work was carried out by identifying the stall zone of an existing axial main fan installation that was used in a ventilation simulation model. This axial fan (400 kW) installation is understood to be capable of operating at up to a pressure of 2300 Pa (at 20º fan blade setting). However, during the simulation studies, it was noted that there was uncertainty about the axial fan performance curve the operating mine had been using. In addition, it was established that the original fan manufacturer did have model test curves for the current axial fan at 20º pitch but the test results for 15º and 25º blade pitch could not be found. A review of ventilation simulation models indicated that they were highly conservative but the project needed more certainty on the main fan performance curve and the stall zone prior to installing a similar second axial fan when introducing a second continuous miner for development. This paper documents the results of the main fan and mine Pressure Quantity (PQ) survey that ascertained the axial fan performance curve and the stall zone and validity of the mine resistance used in ventilation modelling.

BACKGROUND

The fan evaluation work discussed in this paper was carried out at a bord and pillar mine (2 m seam height) with a single Continuous Miner (CM) development. The mine ventilation system consisted of two

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intakes (conveyor, travel) entering from the highwall drawing 110 m$^3$/s using an exhaust fan operating at a pressure of 800Pa (Figure 1).

![Figure 1: Mine ventilation schematic](image)

The main ventilation fan at the mine is an axial flow fan, rated at 75 m$^3$/s at 2,000 Pa and the exhaust fan was operated mostly on the lower pressure part of the fan curve. The return exhaust duct is equipped with dual ducts to accept a second large fan but was fitted with a 45 kW fan instead. However, the mine has operated with only minimal contingency (spare motor) against a fan failure and an adequate backup fan capacity for emergencies. The main fan infrastructure and fan operating point for single fan is shown below (Figure 2) and was not able to provide sufficient capacity for the second CM unit as part of the LW expansion project.

The objective of an independent fan testing was to correlate actual performance to manufacturer’s curves, understand pitch settings of fans, identify the location of stall zone and behaviour of fan in suspected stall zone and enable the selection of operating pitch for twin axial fans in parallel operation. The supplier fan curve were produced from a 1.0 m model fan with an inline fan duct system.

![Figure 2: Mine exhaust fan and its operating point (103 m$^3$/s@ 810 Pa)](image)

PRESSURE QUANTITY SURVEY AND FAN TESTS

A fan test at current pitch (14°) was undertaken to produce fan Pressure-Quantity (PQ); power-quantity curves and to determine the location of the axial fan stall point. An underground PQ survey was conducted to get improved better friction and resistance values for the existing bord and pillar mine and to use these in the simulation model to determine a better estimate of mine resistance. Using this information, the adequacy of the proposed twin fans to meet the required ventilation quantities with a safe margin on stall could then be evaluated and the optimum pitch setting for the fans selected. The current mine resistance determined during a ventilation survey was 0.0669 N.s$^2$/m$^8$. The axial fan test was conducted over three days by a brattice regulator (with a mesh, and Acro-Prop) set-up using the frame of the 70 kPa mine seal in 5 heading outbye of single exhaust airway (Figure 3). The fan was evaluated with ~6 m$^2$ opening at regulator by a pitot traverse at a series of different mine resistances. The regulator orifice sizes were varied to the following predetermined openings, fully closed, 1, 1.2, 1.5, 2, 3, 4, 5, 6, 7 and 8 m$^2$ openings. Measurements were done using the fan Citect instruments in parallel with
Paroscientific barometers and power was measured and logged using the Mines Fluke1375 Power Logger. A separate fan test was carried out at a 4 m² opening at the regulator by a second pitot traverse.

**Figure 3: Location of Brattice regulator for fan stall zone testing**

**MAIN FAN EVALUATION PROCEDURE**

The following procedure was followed prior to the main fan testing and the general test principles can be used when evaluating any other such fans:

- Loosen the bungs on both side of duct at measuring station to enable holding tool for long pitot tube during fan test.
- Construct a substantial prop and mesh stopping frame in main return just outbye of one cut-through probably braced against the 70 kPa seal door frame and seal the spare fan outlet and Y piece explosion panels to reduce leakage.
- Check fan instruments for accuracy and inspect tubing for potential leaks and blockages (test individual static tubes), fan instruments have valves on T piece so check readings can be made while Citect is recording readings.
- Check temperature and vibration transducers and current and voltage Citect readings using clip-on current meter and voltage meter.
- The inside duct, fan blades straightening vein and inlet screen and duct should be cleaned prior to testing.
- Determining fan pitch requires the fan to be stopped and isolated electrically by removing the access cover on the upper platform. The pitch setting on each blade can be determined by measuring in the plane of rotation the distance from the leading edge and the trailing edge of the blade tip from a fixed point.
- All resistance points should have check readings with barometers paralleled with fan pressure instruments.
- Duct DBT and WBT readings should be taken every 30 minutes and atmospheric pressure recorded frequently (at least each time the pitot tube is moved to a different traverse plane).
- Each pitot tube measuring point should be recorded for at least 30 seconds (barometers logging ~ 1 sec, and average data); other points checked including those in stall should be read over ~ 5 minute period noting the accurate time of each reading is essential for later data analysis and photos of each test point orifice regulator hole would help complete the work.
- The stall location will require greater attention to listening to the noise variations and barometers by testing well beyond stall to confirm performance in this region.
- Each fan test duty point test instrument is checked with barometers.
- When using barometers it is important to know the instrument height (RL) (and when tubes connected to instrument the height of the tube inlet) also air density (calculated from abs pressure, WBT and DBT) at instrument and at tube inlet.
- When doing pitot traverses one instrument is used for measuring atmospheric pressure and the other is used to measure static and velocity probe readings. Data logged and time scale on instruments and Citect needs to be the same.
• Both instruments are placed at the same level and the difference between the two pressures plus height correction for the probe ~ from 15 to 45 Pa are made to readings to get pressure values.
• For underground surveys an instrument is left at a known height on the surface, it could be left on the surface outside the control room (non-air-conditioned) at that location WBT and DBT readings are taken.
• The second instrument is taken underground and a synchronised watch is required to record the timing of each reading WB and DB and RL of location required for each reading (if tube is used through a stopping then RL of instrument and tube inlet are required and the WBT/DBT at instrument and tube entry. WBT and DBT readings are taken both inside and outside the duct ~ every 30 minutes for density calculations.
• The instruments should be set up to record an average reading every second and left at each measuring point for at least 1 minute. Data is then averaged for the measuring period.

Figure 4 shows the effect on mine resistance for regulator open area which indicates that the potential stall of the fan at about 1.2 m² open area. The tests were done using a full traverse with pitot tube at current operating point, ~ 4 m² Mine resistance and ~ 2 m² mine resistance point on curve. These positions were expected to provide good control on curve and enable fan instrument calibration to be checked as different flows. The rest of points on curve can be obtained from fan instruments. Barometers were set up in parallel with them to confirm readings. Each of these readings will only take 5 minutes once pressure and flow have stabilised after regulator changes. It is important that stable conditions exist while each set of readings is being taken by ensuring no vehicle movement or opening/closing of man doors and regulator brattice is not flapping.

Pitot grid has 25 points and the probe can reach the middle 3 points from one side and the two outside points are more easily measured with a shorter probe (the long probe is 96”). The fan pressure transducers have a test instrument connection and the barometer is connected to check measurements.

Test Instrumentation
Data was collected for various regulator settings from fully closed to an 8 m² opening using the following instruments:

• The fan instrument (2 X Emerson 3051S1CD Differential Pressure (DP) transmitters one Ranged 0 to 1000 Pa for DP (flow readings) and the other ranged 0 to 3000 Pa for static pressure (at inlet to inlet box)) data from Citect (5 second average data for flow and static pressure).
• Barometer data (one reading atmospheric pressure and the other absolute pressure (alternatively) from each of the fan instrument static rings.)
- Wet Bulb Temperature (WBT) and Dry Bulb Temperature (DBT) readings from inside and outside fan duct.
- Fan shaft power from Fluke1375 Power logger, power data and fan motor efficiency at percentage load data from motor supplier information.
- The two ParoScientific barometers (in working condition and charged)
- A Comark of other DP pressure transducer (if available)
- Quality thermometers for WB, DB readings
- The 3.0 m and 1.5m pitot tubes and tubing
- Electrical equipment to monitor motor current, volts and power factor.
- A platform ladder to do pitot traverse.

**Pitot traverse**

The pitot traverses were conducted on the measuring plane inbye of the inlet to the fan inlet box. Data from the traverse was compared to fan instrument readings from real-time monitor-Citect. Fan instrument readings were checked on barometers connected in parallel with fan instruments (Figure 5 and Figure 6). For pitot traverses, both barometers are placed at the same elevation, one reading atmospherics pressure and the other read the absolute pressure at the pitot tube, static or total, depending on the position of the three-way-valve connecting tubes to the barometer. As both barometers are at the same elevation there is no need to correct differential pressure readings for height differences (although the height effect is taken into account for absolute pressures in density calculations). The pressure reading is simply the difference between the two barometer readings (if there is no zero correction between barometers). The barometer time clocks were synchronised with Citect time and were set to log data every second. Because of a slight difference in logging between barometers (occasional lost data points), “Vlookup” statements are used to synchronise data and to relate 5 second Citect data to barometer readings. Figure 5 shows fan pressure quantity and test instrumentation and measuring planes.

![Figure 5: Fan instrument location and pitot traverse plane and tube ports in duct](image)

Figure 7 shows sample data from three point measurements of static and total pressures, in transitioning between points the pitot tube is rotated ~90° to the flow, this is to give a spike in data to help in differentiating between data points. The average data from ~15 seconds at each point is selected manually from graphs and subtracted from the average atmospheric barometer readings, for the same time period, to produce data for flow calculations.
Table 1 shows the pressure data and flow calculation for the latest pitot tests and an historic test when fan pitch was ~ 21º. At the bottom of tables the average Citect Static Pressure readings from the fan inlet box entry (~ 1 m upstream of the pitot traverse plane on upstream side of the Fan Dampers), the barometer check of these readings and the calculated Citect Flow readings (averaged over the test period). The closeness of these readings (within 2% on pressure and less than 0.5% on flow) shows that fan instruments and Citect readings are within the accuracy range of this type or instrumentation.
Table 1: Summary of pitot traverse results

<table>
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<tr>
<th>Point</th>
<th>SP Static (Pa)</th>
<th>TP Total (Pa)</th>
<th>VP = SP - TP Velocity (m/s)</th>
<th>v = α<em>E</em>(2 * VP/p)^0.5</th>
<th>Q Quantity (m³/s)</th>
<th>ρ Average Duct Density (kg/m³)</th>
<th>FSP Avg SP-VP Calc TP Calc (Pa)</th>
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Average: 1.1946 * 10³ m³/s, 1085.5 Pa, 109.2 Pa, 13.67 m/s;
Citect: 1237.0 C, 0.574 F;
Barometer: 1232.2 K, 0.950;

Summary Results from Pitot traverse results from 26th March 2014:

Average: 1.1946 * 10³ m³/s, 1085.5 Pa, 109.2 Pa, 13.67 m/s;
Citect: 1237.0 C, 0.574 F;
Barometer: 1232.2 K, 0.950;

Summary Results from Pitot traverse results from 17th May 2007:

Average: 1.4546 * 10³ m³/s, 1270.2 Pa, 184.4 Pa, 17.7 m/s;
Citect: 1583.0 C, 0.573 F;
Barometer: 1584.6 K, 0.662;

Average duct density, ρ, 1.131 kg/m³; Duct dimensions at measuring plane (190mm upstream of holes): Height-2.2116 m and Width-3.164 m and Area -6.9974 m²; *summary results from second pitot traverse; **summary results from historic pitot traverse.

DATA ANALYSIS PROCES FROM PITOT TUBE TRAVERSE DATA

Data reduction is partly done using part of the method from AS ISO 5801-2004, Section 27 “Determination of flowrate using a Pitot-static tube traverse” using the formulas from 27.5 for mass flow and expansibility factor, and data from 27.6 below for the flow rate coefficient. The Isentropic Exponent K and density are calculated using ASHREA software “Ashrae LibHuAirPRop” from absolute pressures, WBT and DBT in the duct. The calculation for density from Section 27 using the static temperature method produced exactly the same density as ASHRAE software. The static temperature is calculated to within 0.1º of duct DBT. Given the accuracy of thermometers used during tests, the duct DBT was used in lieu of static temperature in all calculations.

\[ \Delta P_m = \left[ \frac{1}{n} \sum_{j=1}^{n} \Delta P_j^{0.5} \right]^2 \]  

(1)
where
\[ \Delta P_m = \text{Average Differential pressure on measuring plane in Pa} \]
n = Number of points and \( j = \) the identifier for and individual point
\[ Q_m = \alpha \cdot \epsilon \cdot A \cdot \sqrt{2 \cdot \rho_x \cdot \Delta P_m} \] (Mass Flow) kg/s \hspace{1cm} (2)
And
\[ Q = Q_m \cdot \rho_x \] (Flow) m³/s
where
\[ \rho_x = \text{Density at measuring plane} \]
\[ A = \text{Crosssectional Area of measuring plane} \]
\[ \epsilon = \left[ 1 - \frac{1}{2 K} \cdot \frac{\Delta P_m}{P_x} + \frac{(K+1)}{6K^2} \cdot \left( \frac{\Delta P_m}{P_x} \right)^{0.5} \right] \] (Expansibility factor) \hspace{1cm} (3)
\[ \alpha \] is estimated form the Reynolds Number \( (Re_x) \) at the Section and the equation below fitted to data tabulated in section 27.6 of AS ISO 5801-2004 a polynomial fitted to this data was used in calculations.
\[ Re_x = \frac{\rho_x v_x D h_x}{\mu_x} \] (Reynolds at measuring plane) \hspace{1cm} (4)
\[ \alpha = A + B \cdot \text{Exp}(C \cdot Re_x) + D \cdot \text{Exp}(E \cdot Re_x) \] (Flowrate coefficient) \hspace{1cm} (5)

Where
\[ A = 9.92452178341E - 01, \quad B = -5.16985037262E - 03, \quad C = -9.69497112333E - 06 \]
\[ D = -2.63697519572E - 03 \quad \text{and} \quad E = -5.88269194907E - 07 \]
\[ v_x = \text{Average Velocity at measuring plane (m/s)} \]
\[ D h_x = \text{Hydraulic Diamenter at measuring plane (m)} \]
\[ \mu_x = \text{Viscosity of fluid at measuring plane (Pa .s)} \]
The pitot travers data was used to calculate flow and Fan Static Pressure (FSP) and motor shaft power at each regulator setting, and the results were standardised to density of 1.2kg/m³ for use in ventilation simulation models. The pressure, quantity, kW, and efficiency curves for 14º fan pitch in Table 3 and Figure 10 were derived from the test data in Table 2 and Table 3. Figure 8 and Figure 9 show the barometer data using differential pressure rings during fan tests and snapshot of the data for a test regulator area respectively.

\[ \begin{array}{|c|c|c|c|c|c|c|c|c|c|}
\hline
\text{Regulator open area, m}^2 & \text{Flow, m}^3/\text{s} & \text{Static, Pa} & \text{FSP, Pa} & \text{Density, kg/m}^3 & \text{Fan SP at 1.2 kg/m}^3 & \text{Static DP} & \text{Flow, m}^3/\text{s} & \text{Static, Pa} & \text{FSP, Pa} & \text{FSP at 1.2 kg/m}^3 \\
\hline
0 & 38.5 & 2491 & 2474 & 1.123 & 2644 & 102.7 & 2448 & 102.7 & 2448 & 102.7 & 2448 \\
1 & 48.1 & 2132 & 2105 & 1.123 & 2249 & 102.3 & 48.8 & 2112 & 2085 & 2228 & 2228 \\
1.2 & 56.2 & 2101 & 2064 & 1.125 & 2202 & 120.2 & 52.8 & 2082 & 2050 & 2186 & 2186 \\
1.5 & 55.0 & 2104 & 2069 & 1.129 & 2199 & 124.8 & 53.8 & 2091 & 2057 & 2186 & 2186 \\
2 & 72.8 & 2023 & 1961 & 1.130 & 2082 & 248.1 & 75.9 & 2007 & 1940 & 2060 & 2060 \\
3 & 84.0 & 1863 & 1781 & 1.132 & 1889 & 287.0 & 81.7 & 1854 & 1777 & 1884 & 1884 \\
4 & 94.2 & 1468 & 1364 & 1.137 & 1440 & 368.0 & 92.5 & 1455 & 1355 & 1431 & 1431 \\
5 & 97.9 & 1199 & 1087 & 1.140 & 1144 & 413.4 & 98.0 & 1194 & 1081 & 1139 & 1139 \\
6 & 99.3 & 1063 & 947 & 1.142 & 996 & 435.6 & 100.6 & 1022 & 903 & 950 & 950 \\
6.1 & 100.3 & 1045 & 927 & 1.141 & 975 & 433.4 & 100.3 & 1017 & 898 & 945 & 945 \\
7 & 100.2 & 1002 & 884 & 1.142 & 928 & 455.5 & 98.0 & 981 & 981 & 981 \\
8 & 101.5 & 915 & 794 & 1.143 & 833 & 436.8 & 100.7 & 896 & 777 & 816 & 816 \\
\hline
\end{array} \]
Table 3: Fan performance curves for ventilation models at density of 1.2kg/m³

<table>
<thead>
<tr>
<th>Test Fan Data at 14º</th>
<th>Estimated Data At 17.5º</th>
<th>Supplier Fan Data at 20º</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow m³/s</td>
<td>FSP Pa</td>
<td>Shaft kW</td>
</tr>
<tr>
<td>38</td>
<td>2613.7</td>
<td>248.1</td>
</tr>
<tr>
<td>40</td>
<td>2527.6</td>
<td>242.8</td>
</tr>
<tr>
<td>50</td>
<td>2253.6</td>
<td>228.3</td>
</tr>
<tr>
<td>60</td>
<td>2149.9</td>
<td>228.3</td>
</tr>
<tr>
<td>70</td>
<td>2093.4</td>
<td>235.3</td>
</tr>
<tr>
<td>80</td>
<td>1960.7</td>
<td>241.7</td>
</tr>
<tr>
<td>85</td>
<td>1827.3</td>
<td>242.2</td>
</tr>
<tr>
<td>90</td>
<td>1628.6</td>
<td>239.8</td>
</tr>
<tr>
<td>100</td>
<td>973.8</td>
<td>222.0</td>
</tr>
<tr>
<td>108</td>
<td>134.9</td>
<td>191.2</td>
</tr>
</tbody>
</table>

Figure 8: Barometer data from fan test using fan differential pressure rings

Figure 9: Example of data selection method from fan instrument data for 4m² opening
The velocity profile across the duct using data from Table 2 is shown in Figure 11.

CALCULATION OF FLOW FROM FAN INSTRUMENT DIFFERENTIAL PRESSURE READINGS

The origin of differential pressure flow measurements is described below. The Bernoulli equation represents energy conservation for a fluid element:

$$ Const = \rho \cdot g \cdot h + \frac{\rho}{2} \cdot v^2 + P $$  \hspace{1cm} (6)

Where

- $\rho$ = Fluid Density kg/m$^3$
- $v$ = Linear velocity of the fluid element m/s
- $P$ = Pressure in Pa

The first term $\rho \cdot g \cdot h$ is the potential energy coming from height on the gravitational field. For this specific evaluation, constant height of exhaust airflow is assumed, so the equation is re-written to:

$$ Const = \frac{\rho}{2} \cdot v^2 + P $$  \hspace{1cm} (7)

The term $\frac{\rho}{2} \cdot v^2$ is kinetic energy, here the density replaces mass. The pressure $P$ can be understood as a potential energy. Work is stored in compressing the fluid the same way as a compressed spring stores energy.

We apply this equation to a circular cross section pipe that is reduced in diameter as it goes downstream in horizontal direction as in Figure 12.

$$ D \quad P_1 \quad v_1 \quad \frac{v_2}{d} \quad P_2 $$

Figure 12: Flow in Reducing Diameter Pile
\[
\frac{\rho_1}{2} \cdot v_1^2 + P_1 = \frac{\rho_2}{2} \cdot v_2^2 + P_2
\]

(8)

Where the subscripts 1 and 2 represent Upstream and Downstream respectively

As mass is conserved along the pipe

\[Q_M = \rho_2 \cdot v_2 \cdot A_2 = \rho_1 \cdot v_1 \cdot A_1\]

(9)

Where

- \(Q_M\) = Mass Flow in kg/sec
- \(A_r\) = Cross sectional area of pipe upstream
- \(A_2\) = Cross sectional area of pipe downstream

Squaring both sides of (4) and solving for \(v_2^2\) we get

\[v_2^2 = v_1^2 \cdot \left(\frac{\rho_1 + A_1}{\rho_2 \cdot A_2}\right)^2\]

(10)

Rearranging Equation (3)

\[2 \cdot (P_1 - P_2) = \rho_2 \cdot v_2^2 - \rho_1 \cdot v_1^2\]

And substituting \(v_2^2\) from equation (5) into this equation we get

\[2 \cdot (P_1 - P_2) = v_1^2 \cdot \left(\frac{\rho_1 \cdot A_1}{\rho_2 \cdot A_2} - \rho_1\right) = v_1^2 \cdot \frac{\rho_2 \cdot (\rho_1 \cdot A_1)^2 - \rho_1 \cdot (\rho_2 \cdot A_2)^2}{(\rho_2 \cdot A_2)^2}\]

Hence \(v_1\) can be written as

\[v_1 = \sqrt{2 \cdot (P_1 - P_2)} \cdot \sqrt{\frac{(\rho_2 \cdot A_2)^2}{(\rho_2 \cdot (\rho_1 \cdot A_1)^2 - \rho_1 \cdot (\rho_2 \cdot A_2)^2)}}\]

(11)

And

\[Q = v_1 \cdot A_1\] Quantity in m³/s

(12)

Derivation of Flow Calculation in Citect

The \(P_1 - P_2\) (ΔP) in equation 6 above is measured loss between the two static rings and the formula does not allow for shock losses in the inlet Box so the value of \(P_1 - P_2\) needs to have the inlet box shock loss subtracted from it.

Shock Loss, \(P_s = K \cdot \rho_1 \cdot \frac{v_1^2}{2}\)

(13)

Where \(K\) = the shock loss factor

By substitution in Formula 6 above (for simplification) let

\[C = \sqrt{\frac{(\rho_2 \cdot A_2)^2}{(\rho_2 \cdot (\rho_1 \cdot A_1)^2 - \rho_1 \cdot (\rho_2 \cdot A_2)^2)}}\]

Then

\[v_1 = C \cdot \sqrt{2 \cdot (P_1 - P_2 - K \cdot \rho_1 \cdot \frac{v_1^2}{2})}\]

Substituting for \(C\) and \(\Delta P\) in equation

\[v_1 = C \cdot \sqrt{2 \cdot (P_1 - P_2 - K \cdot \rho_1 \cdot \frac{v_1^2}{2})}\]

Squaring both sides and multiplying out

\[v_1^2 = C^2 \cdot 2 \cdot P_1 - C^2 \cdot 2 \cdot P_2 - C^2 \cdot K \cdot \rho_1 \cdot v_1^2\]

Rearranging

\[v_1^2 (1 + C^2 \cdot K \cdot \rho_1) = C^2 \cdot 2 (P_1 - P_2)\]

Separating \(v_1\) and taking Square root of both sides results in

\[v_1 = \sqrt{2 \cdot (P_1 - P_2)} \cdot \sqrt{\frac{(C^2)}{(1 + C^2 \cdot K \cdot \rho_1)}}\]

(14)

In the PLC for Citect Density is assumed to be constant so a fixed Multiplier by \(\sqrt{\Delta P}\) will give Q

Citect Differential Pressure Multiplier \(F\) = \(\sqrt{2 \cdot A_1 \cdot \sqrt{(C^2)}}/(1 + C^2 \cdot K \cdot \rho_1)\)

(15)
With \( \rho_1 = \rho_2 = 1.15, A_1 = 6.974, A_2 = 3.65469 \) and \( K = 1.0 \) (\( K \) is shock loss factor for inlet box referenced to the inlet box entry area and was derived by calculating the value of \( K \) that makes the flow calculated from the differential pressure on fan instruments equal the flow calculated from pitot traverse.

\[ F = 4.82 \]

As Differential Pressure (\( \Delta P \)) comes into Citect as a 4 to 20 milliamp signal ranged from 0 to 1000Pa

The milliamp Differential Pressure signal (\( mA \)) is converted to pressure by the following formula

\[ \Delta P = (mA - 4) \cdot \frac{1000}{16} \text{ in Pa} \quad (16) \]

As the Flow is the Square Root of the Differential Pressure Multiplied by \( F \) the formula in Citect uses a Multiplier of

\[ F \cdot \sqrt{\frac{1000}{16}} = 38.10545 \]

Therefore in Citect \( Q \) (Flow) = \( 38.10545 \cdot \sqrt{(mA - 4)} \) in m\(^3\)/s \quad (17)

Unfortunately the densities \( \rho_1 \) and \( \rho_2 \) are not equal and as flow increase they become less equal and the accuracy of the Citect calculation reduces. An adjustment could be made in Citect to adjust for density based on static pressure, differential pressure and duct temperature which would improve the accuracy. The density values can be calculated by the following formula:

\[ \rho = \frac{P \cdot M}{R \cdot Z \cdot T \cdot 1000} \]

Density (\( \rho \)) = Absolute Pressure (kPa) * Molecular Weight / (\( Z \) * \( R \) * (273.15 + DBT °C))

And as Molecular Weight and \( Z \) (Compressibility Coefficient) will not vary much with condition in fan duct a more accurate flow could be calculated.

It should be noted that in data analysis the Citect \( \Delta P \) values were back calculated using the Citect Flow by reversing the formula to get the \( \Delta P \) i.e. \( \Delta P \) (Pa) = (Flow/4.82)\(^2\). Unfortunately Citect rounds flows to zero decimal places so the \( \Delta P \) values generated are noisier than they should be and accuracy only comes from averaging many values. It would be advisable to report Citect Flow data to 1 decimal place given the accuracy of \( \Delta P \) Values (If only to make future back calculations more accurate) or to store the Actual \( \Delta P \) values as well as flows.

Note Formula 9 was used to recalculate the fan instrument and barometer differential readings for fan test using a shock loss “K” value of 1 was used to determine flows from differential pressure values (Formula 8). The value of 1 was based on results from pitot tests (Tables 1), however it should be noted that the K value determined by the same method from the pitot test previously gave a K value of 0.655, substantial less than the value of 1 determined during this test. However, no error could be found in the data to cause this difference and instrument values matched. It can be suspected that there is some flow disturbance at the inlet box entry (probably associated with inlet dampers and inlet screen) that was affecting results. It should be noted that the manufacture predicted a loss factor (K) of 0.8 for the inlet box.

As a result of this, the Shock Loss factor (K) was recalculated between the pitot measuring plane and the fan inlet static ring (ignoring the inlet box static ring values) and the agreement between data sets was much closer averaging 1.4 and ranging from 1.364 to 1.44 (Note these new values are relative to the velocity pressure at the pitot measuring plane c.f. previous values that were relative to the velocity pressure at the static ring at the entry to inlet box). This provides greater confidence that there is not a calculation error; although there is some flow disturbance around the static ring at the entrance to inlet box affecting results. For this reason the more conservative K value of 1 has been used in determining flow from fan static rings. The inlet box Loss factor K is determined by solving for the value of K that gives the same velocity \( v_1 \) (from formula 14 at entrance to inlet box) to the Velocity \( v_1 \) determined from the mass flow calculated by pitot traverse divided by the area and the density at the entrance to inlet box. The paragraphs above have demonstrated the method of calibrating the shock factor or loss coefficient in the flow calculations in main fan flow readout using differential area measurement techniques.
UNDERGROUND PRESSURE QUANTITY SURVEY

The Pressure Quantity Survey was also conducted with one barometer located on surface and surface WBT and DBT readings were taken every 30 minutes by control room operator. The results of PQ survey were used to calibrate the simulation model to results measured in survey. The original mine model resistance and calibrated resistance using PQ survey were 0.09087 $\text{Ns}^2/\text{m}^8$ and 0.0669 $\text{Ns}^2/\text{m}^8$ respectively. To get a reasonable agreement with the ventilations Survey, adjustments were made to model roadway dimensions to better reflect those measured in survey ~ 0.4 m increase in height was applied to most roadways, the critical overcast resistance was set to the measured value of 0.04663 $\text{Ns}^2/\text{m}^8$ and the K factor (Friction Factor) for smooth blasted type roadways was reduced from 0.012 to 0.007837kg/m$^3$. The changes in these values in the model reduced the mine resistance from 0.06645 $\text{Ns}^2/\text{m}^8$ to 0.03991 $\text{Ns}^2/\text{m}^8$ and increased development face quantities by ~ 15 m$^3$/s over the original model. The PQ survey and the pitot survey provided additional confidence on the validity of the simulation model and its application for the long term project scenarios.

The relative static pressures of each survey point were calculated assuming polytrophic flow using the presented in Chapter 6 of McPherson 2008 and compared with the methods by Hemp in Chapter 6 of Burrows 1989. In reality they are both the same method with a slightly different mathematical arrangement. Chapter 3 of McPherson 2008 gives details. However, evaluation did not result in same results that can be reasoned due to using real moist air densities calculated with ASHRAE LibHuAirPRop software rather than ideal gas densities used in equations.

McPherson Formula

$$F_{12} = \frac{u_1^2 - u_2^2}{2} + (Z_1 - Z_2) \cdot g - R \cdot (T_2 - T_1) \cdot \frac{\ln(P_2/P_1)}{\ln(T_2/T_1)}$$

$$p_{12} = \rho_a \cdot F_{12}$$

$p_{12}$ = Frictional Pressure Drop between stations 1 and 2 (Pa)

$\rho_a$ = Average density between stations 1 and 2 (kg/m$^3$)

$F$ = Work done against friction between stations 1 and 2 (J/kg)

$u$ = average velocity at section (m/s)

$Z$ = height of station above Datum or Reduced Level (m)

$g$ = local acceleration due to gravity (m/s$^2$)

$R$ = Mean gas constant for moist air (J/kg.K)

$T$ = Absolute Temperature at station ($K^\circ$)

$P$ = Barometric pressure at Station (Pa)

Hemps Formula

$$p_{12} = -(P_2 - P_1) - g \cdot \rho_a \cdot (Z_1 - Z_1)$$

$$F_{12} = - \int (vdp - g(Z_1 - Z_1) - g \cdot \int WdZ$$

$v$ = Specific volume (m$^3$/kg) i.e. $1/\rho$

$W$ = Humidity Ratio (kg/kg) (kg water / kg air) in moist air

A summary of these equations is given in paper by a Prosser and Loomis, 2004. Apart from the humidity ratio term and the lack of velocity term in Hemps formula it is mathematically the same as McPhersons Formula for $F_{12}$ (refer Chapter 3 McPherson, 2008 for details). During this underground PQ survey and measured velocity values, when the velocity term was removed from McPherson’s equation the relative static pressure results fall within 0.2 Pa with Hemps pressure equation. Therefore, this suggests that the use of either of the equations is of less significance unless there is large variations in measured velocities between two ventilation stations.
CONCLUSIONS

The main fan tests and underground PQ survey provided the following assurances to the project:

- The main axial fan was tested through a range of operating points beyond the currently perceived pressure stall point of 2.1 kPa. The fan test study validated the fan curves with different pitch setting for use in ventilation simulation studies, which is to give a good operating safety margin from the maximum operating point of 2.6 kPa.

- The study has shown that, when in doubt w.r.t. the main fan curve stall zone, the fan tests provide assurance on the safety margin for the mine fans to operate. Nearly 500 Pa difference in the perceived fan stall point and maximum operating point during field observations for the identified fan pitch was noted.

- The underground PQ survey provides the valuable information in terms of accuracy of mine resistance values and K factors used in the simulation model. The model friction factor and resistance values used were conservative. When remodelled using new friction factor derived from the underground PQ survey, the face quantities were increased by up to 15 m³/s reflecting the operating mine conditions.

- The study has noted that for expansion project decision making, carrying out a desktop based simulation models provide lesser assurance for critical shaft and fan infrastructures decision making.

- Based on the field test in seeking the fan stall zone, it is noted that in this specific case, the stall zone suggested by the supplier was conservative and the main axial fan was at least 500 Pa away from the potential stall point in a simulation model. However, it is not the intention to operate at the maximum pressure. This finding has enabled the operation invaluable additional information in managing the risk and appropriate decision making.

- The paper also demonstrated the method of calibrating the shock factor or loss coefficient in the air flow calculations in main fan flow readout using differential area measurement techniques.

- The underground PQ survey data evaluated using two different methods has shown that the use of either of the equations is of less significance unless there is a large variation in measured velocities between two ventilation stations.

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CURRENT DEVELOPMENTS AND CHALLENGES OF UNDERGROUND MINE VENTILATION AND COOLING METHODS

Amin Kamyar¹, Saiied Mostafa Aminossadati¹, Christopher Leonardi¹ and Agus Sasmito²

ABSTRACT: The mining industry has experienced a dramatic change over the past 20 years in terms of methods and equipment as well as human resource policies. These changes have had impacts on the design of mine ventilation systems. Although feasible developments have been implemented to some extent, in some other areas ventilation planning still requires further improvements to provide a healthy work environment at a reasonable cost. The boom in energy costs has also encouraged mine ventilation designers to seek for efficient use of energy and optimization strategies. The electricity consumption by mine refrigeration plants should be reduced possibly without any adverse effects on the safety of workers. This study presents an overview of the latest techniques used by the experts to address these issues. A revision of the novel ventilation strategies and mine refrigeration methods, and their ultimate effect on efficiency and mining costs would be identified. Finally, likely future developments in the area of mine cooling are outlined.

INTRODUCTION

Australian mines are directed toward deeper underground operations as exploration tools discover orebodies located at great depths. Deeper working environments imply the need for a feasible means of combating thermal pollution. Workers subjected to heat stress experience serious hazards in terms of health, safety, productivity and morale (Brake 2002). In general, there are two common strategies for underground cooling; one is the use of mine ventilation and the cooling effect of the airflow; and secondly use of refrigeration to provide working areas with low-temperature environments. Despite the recent developments in mine air-conditioning, many Australian miners still suffer from thermal discomfort while on duty (Brake 2001a). Thus, higher cooling capacities at deeper levels becomes the major concern of mining companies which imposes higher initial and operational costs. The rise in energy costs on the other side, has made the companies seek for energy management strategies that rectify the inefficiencies of current refrigeration systems or reduce excessive power consumptions. In order to achieve an energy efficient mine cooling system, determination of some factors are essential. These factors include optimum airflow and wet bulb temperature values, applying novel plant components, appropriate integration of components and on-demand plant operation (Marx et al., 2006). Obviously, the implementation of energy efficient projects must not counterbalance the comfortable working conditions for workers. To attain this goal, an exhaustive knowledge of the available technologies and their functionality for different conditions is vital. This paper aims to give an overview of the latest mine cooling strategies being practiced in different mines. The characteristics of these technologies are presented in the form of artificial (refrigeration) methods. In addition, the available optimisation and energy efficiency technologies are outlined along with reported effects from the reviewed case studies. This study will hopefully shed light on the current status of mine ventilation and refrigeration as well as potential energy management techniques for future Australian mining operations.

MINE COOLING STRATEGIES

With the increase in the temperature of the underground working environment, the conditions approach an allowable upper limit called heat stress index. When the conditions surpass this index, operations must be curtailed unless a suitable mine cooling strategy is introduced. There are various heat stress

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indices used worldwide such as dry-bulb temperature and wet bulb temperature of which the latter is quite common since it is appropriate for humid environments as well. For instance in Australian mines, the rule of thumb is that with average temperatures exceeding 28°C wet-bulb (on the hottest days) or the temperature of any working area exceeding 32°C, an underground cooling method must be implemented. Each of the available cooling methods has its own merits and demerits. The satisfactory function of the mine air-conditioning system provides the hot underground mines with the opportunity to exploit deeper reserves and to increase production. Figure 1 shows the hierarchy used by Anglo American operations in terms of different cooling strategies for South African mines. With Australian mines being affected by similar conditions, the trend is to implement the experience obtained for the gold and platinum mines of South Africa. However, the dynamic nature of longwall mining (with frequent moving face) brings about the need for a hierarchy of various strategies for coal mining (Belle and Biffi, 2010).

**Figure 1: Hierarchy of implemented cooling strategies by Anglo American operations for South African mines (Belle and Biffi 2010)**

**Artificial cooling strategies**

The first use of artificial cooling for underground mines goes back to 1860s when heat control was done by transporting blocks of ice underground in ore cars. However, the earliest use of vapour compression refrigeration was in 1921 to cool an underground mine at a depth of about 2400 m at the Morro Velho mine in Brazil (McPherson 2012). In Australia, a 3 MW plant that used chilled water to cool the deep environments with the aid of high-pressure coil heat exchanger was installed at Mount Isa for the first time in 1960s. However, the first surface refrigeration system for an Australian coal mine was in the Bowen Basin at Central Colliery for a depth of 542m about two decades ago (Belle and Biffi 2010).

**Mine Refrigeration plants**

**Surface Bulk Air Cooling**

Currently, surface Bulk Air Cooling (BAC) is the most commonly-used cooling technique in Australian coal mining (Figure 2). The largest surface BAC in Australia operates at Mount Isa with a capacity of 36 MW of refrigeration (Van Baalen and Howes 2009). The vapour compression cycle works via compressing the refrigerant vapour to a high pressure (and high temperature) before sending it to the condenser (a heat exchanger) where it reaches a liquid form. Condensation is done with the aid of cold water coming from cooling towers. High pressure liquid then flows into a receiver followed by an
expansion valve. Upon passing through the valve, the liquid refrigerant experiences an abrupt drop in pressure (along with a dramatic drop in temperature) and sudden expansion (flash off) resulting in the evaporation of the liquid. The low-pressure liquid then flows to the surge drum which separates the liquid and gas phases to ensure only vapour is sent to the compressor. The liquid refrigerant passes through the evaporator (another heat exchanger) where it absorbs the heat from air or water and boils. The vaporized refrigerant then enters the compressor and the refrigeration cycle restarts (McPherson 2012). Efficiency of each of these components affect the Coefficient of Performance (COP) of the refrigeration cycle.

**Figure 2: Schematic of a typical surface Bulk Air Cooling (BAC) refrigeration plant**

Brake (2001b) indicated the key engineering considerations in the design of mine refrigeration plants. The author outlined the major components as screw compressors, refrigerant and plate heat exchangers as evaporator/condenser as well as cooling towers require specific design criteria. Recently, Hooman et al., (2015) presented an inclusive step-by-step guide for proper selection and design of mine BAC heat exchangers in terms of parameters such as size, location, inlet conditions, water loading, environmental conditions and surrounding activities. Lack of a unique design code for mine cooling plants, has prevented most plants from fully exploiting the refrigeration capacity (Brake 2001a). The major challenge encountered when using this technique is to ensure the arrival of acceptable cooled air at the coal face considering the long distance from the plant.

**Underground cooling techniques**

Despite all the advantages of surface BAC, implementing an underground solution for thermal pollution especially for ultra-deep mines becomes essential. Underground refrigeration is usually maintained by: chilled water from surface, secondary cooling of air, recirculation in ventilation districts or tertiary (in-stope) cooling. For longwall and development, the trend is to locate a coil heat exchanger (BAC) in a mined loop. Chilled water is then pumped through the insulated steel pipes (installed boreholes) to the underground coil BAC. A great proportion of the intake air is directed to pass through the BAC chamber with the aid of air locks or auxiliary fans (Belle and Biffi 2010). Figure 3 demonstrates the schematic of the overall design of such a system studied by O'Connor et al., (2013) to evaluate the feasibility for a Bowen Basin mine. On the right hand side, location of the heat exchanger is shown relative to the
longwall panel. Comparing the positional efficiency of the various methods (surface BAC without underground cooling; surface BAC with underground cooling and underground cooling only), the authors found that utilising the underground heat exchanger alone is the best option.

Figure 3: Schematic of an underground cooling system and location of the BAC for a Bowen Basin mine (O’Connor et al., 2013)

With the increase in the mining depth and the pumping distance, the operational cost of an underground BAC also rises. Locating the refrigeration plant underground, is another method that shortens the distance in which chilled water (air) travels. An example of such systems is presented in Figure 4. This technique was studied by Ramsden et al., (2007) and compared with surface cooling systems. The major limitation regarding the use of this type is the insufficient means of heat rejection which is the return air only. While surface plants reject the heat to the general ambient air, the limited hot air flow in the intake airways has adverse effects on the efficiency of underground plants. This is along with higher comparative costs due to the needs for high-pressure compressors, extra excavation and installation (van den Berg et al., 2013). For coal mines, another challenge stems from the frequent movement of equipment with longwall as compared with metal mining.

Thus, in some cases the underground refrigeration system consists of two components: the main refrigeration plant chamber/chillers as well as the movable component which comprises air coolers (coil heat exchangers) with a water distribution network. Some other mines use a “hybrid” system which is a mixture of surface BAC and underground refrigeration with BAC located in critical locations. For instance, this system was planned for a block cave operation copper mine in Arizona, USA including surface BAC with a capacity of 105.2 MW and underground air coolers providing 38.5 MW (Bluhm et al., 2014).

Mobile localized (spot) cooling

When the evaporator of the refrigeration unit is in direct contact with the airflow at the place where cooling is required (ie face cooling), the absorbed heat in the condenser is also dumped directly to the return air. Benefits of this system is the immediate cooling at the spot and no loss in efficiency due to water reticulation. Spot coolers work on the same basis as domestic air-conditioning units except that the heat is rejected to the outside atmosphere (McPherson, 2012). District cooling has been
implemented in South Africa since 1950s with capacities ranging from 100 kW to 500 kW. The use of mobile spot coolers is a prevailing strategy for cooling German coal mine longwall faces as well. This technique is likely to be used in Australian mines with the increase in mining depths. However, the challenge is the necessity for an approval for the operation of such coolers in Australian mines (Belle and Biffi, 2010).

Due to the large energy consumption of conventional vapour-compression cooling systems, some alternatives are introduced for spot cooling in mines. One of these techniques is the use of vortex tubes. This device, invented by Ranque (1933), works based on the fact that if the vortex motion of a fluid is confined in a cylindrical tube a significant temperature separation occurs causing one end of the tube to cool down and the other to warm up. This phenomenon takes place without the help of any moving parts. Despite the relatively low refrigeration capacity of vortex tubes, they can be a potential candidate for underground district cooling for many reasons such as light weight, usable when electricity is not available, low initial cost, instantaneous operation and no need for expert operator (Ameen, 2006). Figure 5 delineates the overall system and detailed structure of a vortex tube. Upon injection of the high pressure fluid, a major part of the fluid rotates and moves forward along the periphery of the tube. However due to the nature of the fluid dynamics, the inner part of the flow returns toward the cold exit. As a result of a pressure gradient created by the forced vortex, a cold core is formed near the injection leading to temperature drops (Xue et al., 2013).

**Figure 4: Schematic of underground refrigeration plant (Ramsden et al., 2007)**

**Figure 5: (a) A vortex tube refrigeration system (Ameen, 2006) (b) flow structure inside a vortex tube (Xue et al., 2013)**
Although the function of a vortex tube seems different from regular cooling systems, it can be analysed as a classic thermodynamic cycle to analyse the temperature, pressure and velocity profiles (Ahlborn and Gordon, 2000). The feasibility of using vortex tube refrigeration for underground cooling was investigated by Jinggang et al. (2009) for the first time. The authors indicated the beneficial application of this technique due to some reasons such as: making use of compressed gas underground, possibility of moving the system with the mine working face, no need for long pipping, major cost saving and overall reduction in greenhouse gas emissions. The work on this type of cooling strategy is still immature and should be extended for future potential mining applications. Another novel proposal to provide district cooling in underground environments is the use of high-pressure water as the driving fluid in an ejector refrigeration system (Butterworth and Sheer, 2007). Figure 6 illustrates the schematic of a water vapour refrigeration unit and its major components. The working principle of an ejector is based on gas (vapour) extraction from a space via discharging a motive fluid (water). The fluid exits as a jet through a nozzle acting as a vacuum pump that draws the gas (vapour) into the diffuser and mixing tube. Transfer of energy between the water jet and the low-energy vapour, leads to the mixing of fluids resulting in condensation of a great part of the vapour (Raynerd, 1987). Upon cooling the water to a certain temperature, if the pressure is adjusted to a value lower than the saturation pressure of that temperature, water will start to boil. Due to the high enthalpy of vaporization of water, a small mass fraction (0.17%) should be evaporated in order to have a reduction of 1°C in temperature. Steam-jet ejectors can provide the low pressures required for this purpose. The ultimate result is the removal of heat from the evaporator section by the motive water jet. Butterworth and Sheer (2005) investigated the potential functionality of such systems for underground cooling using the available mine water. It was concluded that this technology can be implemented for backfill cooling. The backfill usually imposes a heat load on the ventilation itself. Installing a cooling ejector system on the levels above the stopes, a high-pressure water jet will reduce the temperature of the backfill efficiently.

**Figure 6: Schematic of an ejector for water vapour refrigeration (Butterworth and Sheer, 2007)**

*Ice Cooling Systems*

The use of ice from the surface for underground cooling dates back to 1927 but was found to be inefficient and infeasible (Gebler 1980). Later, South African Mponeng mine tested the use of ice to provide cooling at a depth of 4km with rock temperatures reaching 55°C. Compared to chilled water, with a reduction of 70% in the mass flow rate the same cooling capacity can be provided using melting ice. The initial, operating and maintenance costs of ice-producing power plants are high. For the case of South African mines, the advantages of running such systems outweighs the cost-imposed burdens when a pumping head of 2500 m is reached. This can be the case for future Australian ultra-deep mines.
as well (Belle and Biffi 2010). When water is pumped underground, its temperature goes up as a result of its potential energy being converted to heat. As indicated by Kidd (1995), for the case of Vaal Reefs mine, an increase of about 2.4°C per 1000 m of pipe-run were measured. Whereas for ice slurry, the mixture temperature remains constant due to melting of ice. In another study by Ophir and Koren (1999) the application of ice slurry for underground cooling at the Western Deep Level Gold Mine, South Africa was described. An ice slurry plant comprised of four 3 MW units was used to produce the ice slurry transported to depths of 4000 m with the aid of gravity. Mackay et al., (2010) carried out a comparative modelling to specify the break-even depth at which each refrigeration mode should be applied for Impala Platinum mine, South Africa. They discovered that despite the excessive growing trend of required flow rate of chilled water, the ice flow rate for the same refrigeration capacity is still affordable at lower depths. It was also indicated that in terms of cost analysis, after a depth of 2900 m the results are in favour of ice cooling method. It is noteworthy that ice can also be produced for other purposes than underground cooling, which is the thermal energy storage and load shifting capability. These systems are particularly worth implementing where power tariffs are levied such that substantial savings can be achieved by producing ice at night (off-peak periods) and using it during daytime at peak tariff times. In terms of power cost saving, this technique might not be profitable for Australian mines due to fixed tariffs; however, it could help to install a smaller refrigeration plant by load profiling where there are electricity constraints (van den Berg et al., 2013). For this application, the refrigeration plant consists of two components: primary (base load) machine and thermal storage dam. Chilled water exits the former and then enters the latter which contains tube banks. Glycol passes through these tubes and causes a layer of ice to be formed during low-temperature periods throughout the day. The formed ice then adds to the cooling capacity by melting during hotter periods of operations (Bluhm et al., 2014). Figure 7 shows the schematic of such a system.

Figure 7: Schematic of refrigeration plant integrated with ice thermal storage (Ramsden et al., 2007)

Aside from the mentioned novel methods, seasonal thermal energy storage has found its way to the mining industry, recently. In the summer, heat is stored within the rock-pit to be used later on for heating in the winter, while the “cold” energy is captured and stored in the rock-pit for cooling in the summer. This has been utilized as a “natural heat exchanger” at some areas such as Creighton mine in Canada. Seasonal ice thermal storage is another method which includes the converting the warm service water into ice by spraying it into the incoming sub-zero ventilation air in winter. This technology used in Stobie mine in Canada, then uses the stored ice in summer to produce chilled water. Mining industry should be more aware that renewable energy can be harvested and used for mine cooling which can lead to energy savings, carbon footprint and cost reduction.
OPTIMIZATION AND ENERGY SAVING METHODS

Monitoring and Control

With the advent of automation technologies, it is nowadays possible to implement an accurate monitoring and control technique to observe the conditions of air temperature and velocity, contaminants and water flow. Real-time Energy Management System (REMS) is currently drawing more attention as a tool providing an optimized schedule for refrigerating the hot underground areas (Webbeer-Youngman 2005). The purpose is to reduce the energy consumption of mine cooling with the aid of technologies such as Variable Speed Drives (VSD), control valves and other Demand-Side Management (DSM) methods. Pelzer et al., (2010) implemented this strategy for three South African mines to monitor and optimize the inlet chiller temperature. They reported a value of 32 416 MWh reduction in electricity consumption due to the increase in the Coefficient of Performance (COP) of the plant. The application of the same monitoring system was also reported by Vosloo et al., (2012) for a water reticulation system at Kopanang mine, South Africa. The authors claimed that a 2% reduction in overall power consumption corresponding to an annual cost saving of US$ 636 400. In another study by du Plessis et al., (2013a) the outcome of implementing various energy saving strategies for the Kusalethu mine in South Africa was reported. The applied strategies included evaporator and condenser flow control using VSD, BAC water flow control using valves and retrofitting of old pre-cooling towers. The implementation of these strategies was found to result in a saving of about 31% in the total plant power consumption while keeping the refrigeration and ventilation requirements met and the COP of the cooling system enhanced overall (See Figure 8). As one of the techniques, the effect of VSDs was investigated by du Plessis et al., (2013b) for 20 South African mine cooling systems. It was concluded that a power savings of about 168 633 and 144 721 MWh/year is achievable by installing VSDs on chiller compressor motors and all pump motors, respectively. Mare et al., (2015) also explored the effect of energy saving strategies by varying the flow according to the demand for two mine case studies. VSDs were installed on the evaporator, condenser and BAC pump motors. For the evaporator, the strategy helped to monitor and control the dam level while for the condenser it provided a fixed temperature difference within the condenser vessel. Reduction of full-load conditions for the BAC and a fixed constant wet bulb temperature (8°C) was also attained. Beside these merits, the major barriers of using VSDs are mainly indicated as technical (non-linear loads), economic (high price) and awareness (personnel scepticism about achievable energy saving) issues. By addressing these issues, implementation of VSDs for Australian mine cooling system might be more common in future.

![Figure 8: COP of the cooling system before and after implementing the energy saving strategies (du Plessis et al., 2013a)](image)

In recent years, another strategy has been instigated in Chinese mines which is based on extracting the cold from the underground water inrush (a phenomenon in which water resources suddenly fill the space of the mine during mining processes) to cool the warm airflow (Gao and Liu 2009). In summer, High-temperature Exchange Machinery System (HEMS) is able to provide cooling to the working environment as well as buildings. The system also helps to provide the extracted heat from water inrush...
for buildings and showers in winter (Ping et al., 2011). Figure 9 shows the layout of this system comprising of closed-loop water as well as open-loop air circulation systems. Since the refrigeration plant (chilling water) and air cooling station are located at different levels, a pressure transition is also installed to reduce the pressure when reaching lower levels. Qi et al. (2011) reported the operational effects of this system for the Sanhejian coal mine, China for a depth of 1000 m. They indicated that the airflow temperature at a point of the working face decreased from 38°C to 30°C. This method is claimed to bring about environmental benefits and economic sustainability in addition to energy management. A similar thermal energy management system was introduced by Niu (2015) where a heat recovery system was utilised during winter. A two-stage cooling system was proposed that made use of the lower underground cooling requirements in winter, to run the heat pumps in heating conditions by recovering heat from the low temperature mine return air and mine water.

![Figure 9: Schematic of HEMS cooling system (Manchao et al., 2010)](image)

CONCLUSIONS

This paper presented an overview of the common trends in mine ventilation and refrigeration as well as more recent energy efficiency and optimization practices in underground cooling. Despite all the recent developments in South African mines, many Australian mines are still lacking a mining code of practice for heat management in mines. If the sources of heat in the underground environment are measured in an accurate way, a suitable cooling strategy can be proposed which is an inevitable fact with the increase in depth of mines in future. Energy efficiency and optimization strategies will address the concerns of management in terms of functionality and cost-effectiveness of these strategies. Introduction of novel monitoring systems, implementing control strategies to obtain cooling-on-demand, thermal energy recovery from available resources and improving the performance of current cooling plants are the main ways to achieve this goal. Previous case studies acknowledge that large financial benefits can be acquired if refrigeration plant of a mine is optimised.

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Applied Modelling of Ventilation and Gas Management to Increase Production in a Single Entry Longwall Panel

Dennis Black

ABSTRACT: Mine design and operating practices in European countries may differ significantly from those accepted in Australian mines. This paper describes work carried out to review longwall productivity and gas management at one European mine extracting multiple seams 900-1000 metres below the surface. The longwall panel that is the subject of this paper is extracting a 215 metre wide face in the 1.5 metre thick B seam, which is extracted prior to the main 3.0 metre thick A seam to assist with degassing. Gateroads developed to access the B seam longwall are single entry, developed using roadheaders and supported with steel arches. Additional timber cog support is installed in an attempt to reduce roadway closure due to abutment loading. Due in part to statutory limits on gas concentration and air velocity, longwall production at this mine is presently restricted to an average rate of approximately 700 t/d.

Mine management were reluctant to consider significant mine design changes, such as two heading gateroads, therefore the investigation focused on determining the extent to which ventilation and gas emission impacted on longwall production performance and recommending actions to improve ventilation and gas management to support increased longwall production.

Ventilation and gas emission modelling was used to evaluate the impact of changes to the longwall ventilation arrangement and partial pre-drainage of the B seam. Modelling demonstrated that increase in longwall production rate to 2200 t/d is easily achievable. A range of additional actions that support further increases in longwall productivity are listed.

INTRODUCTION

In 2014/15, PacificMGM completed a review of longwall productivity and gas management at a European longwall mine extracting multiple coal seams at depths in the order of 900-1000 metres below the surface. Three coal seams are present in the mining zone and the in situ gas content of these coal seams is reported to be greater than 20 m$^3$/t. Mine management advised that the permeability of the B seam is very low and previous attempts to pre-drain the working seam had been unsuccessful.

The mine utilises the longwall method to extract the 3.0 metre thick A seam and 1.5 metre thick B seam. The A seam is separated from the B seam by approximately 20-25 metres of interburden. Extraction of the B seam is sequenced in advance of the A seam, with goaf formation and fracturing above the B seam used to stimulate gas emission and reduce the gas content of the A seam prior to mining. Details of the stone and strata layers above and below the B seam are listed in Table 1.

The B seam longwall panel, LW1, was the focus of this study to evaluate ventilation and gas management practices and identify opportunities to increase longwall productivity. Figure 1 shows the location of LW1 relative to older B seam workings.

The length of the LW1 panel is 1000 metres and the width of the longwall face is 215 metres. The layout of the LW1 and LW2 panels are shown in Figure 2. Single entry gateroads are developed using roadheaders and ground support is in the form of steel arches. Active ground support systems, such as roof bolts and cable tendons, are not routinely used in B seam operations. Timber cogs are installed in

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the maingate roadway, inbye the longwall face, in an attempt to reduce closure due to abutment loading. This roadway typically experiences at least 30% reduction in cross-sectional area. The installation of timber cogs also increases both the resistance and air velocity in this section the longwall ventilation circuit.

The layout of the longwall panel ventilation comprises intake air in both the single entry maingate and tailgate roadways, homotropal flow across the longwall face, and the return air exits the panel via the maingate companion road, inbye the longwall face. A small flow also exits the panel from the inbye tailgate corner of the goaf, adjacent to the installation face. The layout of the LW1 panel ventilation circuit is shown in Figure 2.

**Table 1: Stone and coal layers above and below the LW1 working seam – B seam**

<table>
<thead>
<tr>
<th>Layers Above Roof</th>
<th>Coal Mine Strata and Gas Sources</th>
<th>Depth to Roof (m)</th>
<th>Depth to Floor (m)</th>
<th>Thickness (m)</th>
<th>Ref. Density (t/m³)</th>
<th>Measured Gas Content (%/Vol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7. Siltstone</td>
<td></td>
<td>910.3</td>
<td>912.0</td>
<td>1.7</td>
<td>2.5</td>
<td>1-2</td>
</tr>
<tr>
<td>6. Coal seam – Working Seam (A)</td>
<td></td>
<td>912.0</td>
<td>915.0</td>
<td>3.0</td>
<td>1.3</td>
<td>20-25</td>
</tr>
<tr>
<td>5. Argillite</td>
<td></td>
<td>915.0</td>
<td>916.6</td>
<td>1.6</td>
<td>2.5</td>
<td>1-2</td>
</tr>
<tr>
<td>4. Siltstone</td>
<td></td>
<td>916.6</td>
<td>919.6</td>
<td>3.0</td>
<td>2.6</td>
<td>1-2</td>
</tr>
<tr>
<td>3. Sandstone</td>
<td></td>
<td>919.6</td>
<td>934.0</td>
<td>14.4</td>
<td>2.6</td>
<td>0.05-0.3</td>
</tr>
<tr>
<td>2. Siltstone</td>
<td></td>
<td>934.0</td>
<td>935.6</td>
<td>1.6</td>
<td>2.6</td>
<td>1.0</td>
</tr>
<tr>
<td>1. Argillite</td>
<td></td>
<td>935.6</td>
<td>935.9</td>
<td>0.3</td>
<td>2.5</td>
<td>3-7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Layers Below Floor</th>
<th>Coal seam – Working Seam (B)</th>
<th>Depth to Roof (m)</th>
<th>Depth to Floor (m)</th>
<th>Thickness (m)</th>
<th>Ref. Density (t/m³)</th>
<th>Measured Gas Content (%/Vol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Argillite</td>
<td></td>
<td>937.4</td>
<td>942.3</td>
<td>4.9</td>
<td>2.6</td>
<td>1-2</td>
</tr>
<tr>
<td>2. Sandstone</td>
<td></td>
<td>942.3</td>
<td>949.7</td>
<td>7.4</td>
<td>2.6</td>
<td>0.05-0.3</td>
</tr>
<tr>
<td>3. Siltstone</td>
<td></td>
<td>949.7</td>
<td>951.6</td>
<td>1.9</td>
<td>2.5</td>
<td>1-2</td>
</tr>
<tr>
<td>4. Coal seam (C)</td>
<td></td>
<td>951.6</td>
<td>951.9</td>
<td>0.3</td>
<td>1.3</td>
<td>20-25</td>
</tr>
</tbody>
</table>

**Figure 1: Location of LW1 relative to older B seam mine workings**

Gas emission from the longwall face and goaf contaminates the ventilation air as it passes through the longwall circuit. A roadway connection exists between MG2 and MG1 which directs additional air to the longwall return that assists in diluting gas emissions to maintain CH₄ concentration below the statutory limit.

Statutory limits on CH₄ concentration are similar to Australia with a maximum permissible CH₄ concentration of 1.0% CH₄ on the longwall face and 2.0% CH₄ in the panel return. Local mining regulations also limit air velocity in working places to a maximum 4.0 m/s.
Figure 2: Layout of LW1 and LW2 panel and ventilation circuit in the new B seam mining area

There is presently minimal CH$_4$ contamination of intake air entering the LW1 panel via the tailgate. This is due to LW1 being the first longwall block in the new mining area and the tailgate intake roadway passing through solid coal and is not connected to an adjacent goaf. Due to the nature of the single entry design there is a high risk of CH$_4$ contamination of the tailgate intake air in future longwall panels.

To identify and assess the significance of source of gas emission in the longwall panel, a number of measurement locations were identified where ventilation and gas concentration measurements were taken. The measurement locations are shown in Figure 3 and the results are listed in Table 2.

Figure 3: Ventilation and gas concentration measurement locations in LW1 panel

Table 2: Ventilation and gas concentration measurements recorded in LW1 panel

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>S = Area (m$^2$)</td>
<td>0.8</td>
<td>5.7</td>
<td>12.4</td>
<td>11.5</td>
<td>5.3</td>
<td>10.2</td>
<td>16.5</td>
<td>12.7</td>
</tr>
<tr>
<td>V = Velocity (m/s)</td>
<td>1.5</td>
<td>2.5</td>
<td>1.6</td>
<td>0.5</td>
<td>2.4</td>
<td>1.7</td>
<td>0.6</td>
<td>2.2</td>
</tr>
<tr>
<td>Q = Quantity (m$^3$/s)</td>
<td>1.2</td>
<td>14.1</td>
<td>19.6</td>
<td>5.4</td>
<td>12.9</td>
<td>17.2</td>
<td>10.2</td>
<td>28.3</td>
</tr>
<tr>
<td>Q = Quantity (m$^3$/min)</td>
<td>71</td>
<td>844</td>
<td>1174</td>
<td>323</td>
<td>772</td>
<td>1031</td>
<td>614</td>
<td>1697</td>
</tr>
<tr>
<td>C = CH$_4$%</td>
<td>2.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.9</td>
<td>0.7</td>
<td>0.1</td>
<td>1.3</td>
</tr>
<tr>
<td>Gas Make (L/s)</td>
<td>24</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>116</td>
<td>120</td>
<td>10</td>
<td>368</td>
</tr>
<tr>
<td>Gas Make (m$^3$/min)</td>
<td>1.4</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>6.9</td>
<td>7.2</td>
<td>0.6</td>
<td>22.1</td>
</tr>
</tbody>
</table>

The average production rate in LW1 was approximately 700 t/d, which was largely due to gas concentrations and ventilation air velocity reaching permissible limits. The results of the ventilation survey, presented in Table 2, confirm the CH$_4$ concentration of 0.9% recorded on the longwall face (Location 5) is presently constraining production in the current ventilation circuit design. With air velocity...
of 2.5 m/s recorded at Location 5, there is capacity to increase air flow across the longwall face to dilute face gas emission and reduce CH₄ concentration, however to increase ventilation quantity to the longwall district, action must be taken outbye to increase mine ventilation efficiency, which was determined to be approximately 40%.

Longwall production is also inherently constrained by process tasks specific to the single entry mine design. One significant task in the mining process requires (a) removing the block-side legs of each steel arch support to allow the longwall face to pass, and (b) reattaching the legs to the maingate arches, inbye the longwall face. Timber cogs are also installed inbye the longwall in an attempt to resist closure of the maingate airway due to abutment loading, as this roadway must remain open and serviceable as the main return for the current longwall (LW1) and tailgate intake in the next longwall (LW2).

Details of ventilation air quantity, air velocity and CH₄ emission recorded at the various survey points in LW1 are presented in Figure 4.

Analysis of CH₄ concentrations in ventilation air and the gas drainage system determined the specific gas emission rate in LW1 was 77.5 m³/t. Of this total emission rate, 29.5 m³/t (38%) was released into the mine ventilation system and 48.0 m³/t (62%) was extracted through the goaf drainage system.

Figure 4: Summary of ventilation and CH₄ gas emissions recorded in the LW1 ventilation network

VENTILATION AND GAS EMISSION MODELLING

A ventilation model was developed to simulate current ‘base case’ ventilation air flow and gas emission rates, similar to those measured during the ventilation survey. Including the base case, five (5) models were developed to assess the impact of (a) increasing total air quantity (m³/s) in the longwall panel, (b) modifying the panel design and ventilation circuit to increase ventilation and gas emission management capacity through the addition of a MG Bleeder Road, and (c) partial pre-drainage of the B seam to decrease gas emission during longwall extraction. Values of ventilation air quantity (m³/s), air velocity (m/s) and gas concentration (%CH₄) were recorded at the eight (8) locations in the longwall ventilation circuit of each model. Modelling an increase in longwall production rate is represented by increasing the rate of gas emission from the longwall face (LW Production Gas Emission).

A summary of key assumptions and details of air quantity, air velocity and CH₄ concentration recorded at the eight (8) measurement locations in each model are listed in Table 3. Details of the longwall panel design assumptions in each of the five (5) modelled scenarios are summarised below:

- Model 1: (Base Case) Combined MG and TG ventilation = 25 m³/s, B seam gas content = 11.0 m³/t, average longwall production rate = 700 t/d, Background CH₄ emission rate = 45 L/s, LW production CH₄ emission rate = 90 L/s, Goaf gas CH₄ emission rate = 400 L/s. Airflow (m³/s) and gas emission (L/s) results from Model 1 are presented in Figure 5. This model shows that air
velocity is 60% of capacity and the CH\textsubscript{4} concentration is approaching the 1.0% maximum permissible limit.

- Model 2: Current ventilation arrangement, assessing the impact of increasing LW production rate. Combined MG and TG ventilation = 25 m\textsuperscript{3}/s, B seam gas content = 11.0 m\textsuperscript{3}/t, average longwall production rate = 1050 t/d, Background CH\textsubscript{4} emission rate = 45 L/s, LW production CH\textsubscript{4} emission rate = 135 L/s, Goaf gas CH\textsubscript{4} emission rate = 400 L/s. The CH\textsubscript{4} concentration on the longwall face in this model exceeds the permissible limit.

- Model 3: Assessing the impact of (a) increasing LW panel ventilation air quantity, and (b) increasing LW production rate. Combined MG and TG ventilation = 39.7 m\textsuperscript{3}/s, B seam gas content = 11.0 m\textsuperscript{3}/t, average longwall production rate = 1400 t/d, Background CH\textsubscript{4} emission rate = 45 L/s, LW production CH\textsubscript{4} emission rate = 180 L/s, Goaf gas CH\textsubscript{4} emission rate = 400 L/s. Whilst the air velocity and gas concentration on the longwall face are within permissible limits, the air velocity in the maingate return exceeds the permissible limit.

- Model 4: Assessing the impact of (a) modifying the ventilation circuit to include a dedicated goaf bleed (MG Bleeder Road), (b) increasing LW panel ventilation air quantity, and (c) increasing LW production rate. Combined MG and TG ventilation = 38.9 m\textsuperscript{3}/s, B seam gas content = 11.0 m\textsuperscript{3}/t, average longwall production rate = 2200 t/d, Background CH\textsubscript{4} emission rate = 30 L/s, LW production CH\textsubscript{4} emission rate = 178 L/s, Goaf gas CH\textsubscript{4} emission rate = 400 L/s. Airflow (m\textsuperscript{3}/s) and gas emission (L/s) results from Model 5 are presented in Figure 6. This model highlights the significant increase in longwall production capacity that may be achieved through partial pre-drainage of the B seam prior to longwall extraction.

- Model 5: Assessing the impact of (a) modifying the ventilation circuit to include a dedicated goaf bleed (MG Bleeder Road), (b) increasing LW panel ventilation air quantity, (c) pre-drainage to reduce B seam gas content, and (d) increasing LW production rate. Combined MG and TG ventilation = 38.9 m\textsuperscript{3}/s, B seam gas content = 7.0 m\textsuperscript{3}/t, average longwall production rate = 2200 t/d, Background CH\textsubscript{4} emission rate = 30 L/s, LW production CH\textsubscript{4} emission rate = 178 L/s, Goaf gas CH\textsubscript{4} emission rate = 400 L/s. Airflow (m\textsuperscript{3}/s) and gas emission (L/s) results from Model 5 are presented in Figure 6. This model highlights the significant increase in longwall production capacity that may be achieved through partial pre-drainage of the B seam prior to longwall extraction.

Table 3: Results of ventilation and gas emission modelling to evaluate ventilation design and LW production improvements

<table>
<thead>
<tr>
<th>Modelled Scenario</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
<th>Scenario 5</th>
</tr>
</thead>
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<tr>
<td>Measurement/Location</td>
<td>Vent Air Quantity (m\textsuperscript{3}/s)</td>
<td>Vent Air Quantity (m\textsuperscript{3}/s)</td>
<td>Vent Air Quantity (m\textsuperscript{3}/s)</td>
<td>Vent Air Quantity (m\textsuperscript{3}/s)</td>
<td>Vent Air Quantity (m\textsuperscript{3}/s)</td>
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<td>Background CH\textsubscript{4} Emission Rate (L/s)</td>
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<tr>
<td>LW Production CH\textsubscript{4} Emission Rate (L/s)</td>
<td>90</td>
<td>115</td>
<td>180</td>
<td>180</td>
<td>178</td>
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<tr>
<td>Goaf Gas CH\textsubscript{4} Emission Rate (L/s)</td>
<td>400</td>
<td>400</td>
<td>400</td>
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<tr>
<td>Average B seam CH\textsubscript{4} Production (L/s)</td>
<td>700</td>
<td>1050</td>
<td>1400</td>
<td>1400</td>
<td>2200</td>
</tr>
<tr>
<td>Average B seam CH\textsubscript{4} Concentration (ppm)</td>
<td>11.0</td>
<td>11.0</td>
<td>11.0</td>
<td>11.0</td>
<td>7.0</td>
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</tbody>
</table>
VENTILATION IMPROVEMENTS TO SUPPORT INCREASED LONGWALL PRODUCTION

The aim of the recommended changes to the longwall ventilation circuit is to draw goaf gas emissions away from the longwall face and maingate return, and confine the gas, as far as practicable, in the goaf and maingate bleeder roadway. The design partially segregates the maingate return roadway from the goaf and directs ventilation air to the bleeder and main return to dilute methane concentration below the legal limit.

Modelling has shown that developing an additional roadway connection between the main return airway and the longwall install face (MG Bleeder Road) and changing the ventilation arrangements in the longwall panel to direct return air to the MG Bleeder Road, as indicated in Figure 7, allows total ventilation air quantity to the longwall district to be increased, while remaining compliant with prescribed maximum air velocity limits. The improved ventilation arrangement also has a positive effect on reducing CH₄ concentration in the maingate return airway as the low pressure point in the ventilation circuit is located at the MG Bleeder Road which draws CH₄ emission from the goaf away from the maingate. Through implementing the recommended changes to the design of the longwall ventilation circuit, modelling indicates a potential 100% increase in longwall production rate.
GAS DRAINAGE IMPROVEMENTS TO SUPPORT INCREASED LONGWALL PRODUCTION

The mine utilises goaf drainage to extract gas from the goaf which reduces the volume of gas that would otherwise contaminate the mine ventilation system. Boreholes are drilled into the goaf from drill sites located behind the retreating longwall face. The layout of the goaf drainage boreholes in the LW1 goaf are shown in Figure 8.

The maximum length of the 48 boreholes drilled into the goaf was 120 metres and the average length was 80 metres. Drilling goaf drainage boreholes in the single entry longwall return, behind the retreating longwall face, results in the full length of the borehole, including casing, being drilled in fractured ground. In such conditions it is difficult to effectively seal the borehole to minimise leakage. The average concentration of gas present in the goaf drainage system was 28% CH₄ / 72% Air which highlights significant leakage.

The current goaf drainage program does have a positive impact on reducing gas emissions into the ventilation system however changes are required to increase the efficiency and effectiveness of the gas.
drainage program (Black and Aziz 2011). Modelling has demonstrated that reducing the gas content of the B seam by 4.0 m$^3$/t, in addition to modifying the longwall ventilation circuit, will support increasing longwall production to 2200 t/d. Expanding the gas drainage program to increase the volume of gas extracted by pre-drainage and goaf drainage will support further increases in longwall production.

Given the layout of the longwall panel allows air to be drawn into the goaf and the recommended change to the ventilation circuit includes ventilating the perimeter of the goaf, it is important that relatively low suction pressure is applied to goaf drainage bores to reduce air contamination. Therefore to increase total goaf gas extraction it will be necessary to increase the total number of boreholes available. Six (6) areas, indicated in Figure 9, have been identified where gas drainage bores should be drilled to increase total gas extraction. A summary of the five (5) goaf drainage targets (A – E) and one (1) pre-drainage target (F) is provided below:

A. Continue drilling bores into the goaf from drill sites along the main gate return. Increased total gas extraction and reduced air contamination will be achieved by (a) improved sealing around longer standpipes, (b) reducing the suction pressure applied to each borehole, and (c) increasing the total number of active boreholes.

B. Maintain active gas extraction from the goaf above the installation face position of the previous longwall panel.

C. Maintain gas extraction from the goaf of the previous longwall block. Gas drainage range to be maintained in the tailgate and connected to goaf drainage bores. Regulate flow from each borehole to maintain high CH$_4$ concentration.

D. Drill additional bores into the roof above the longwall installation face and connect to the main gas drainage range using a pipeline that extends through the tailgate seal. Regulate flow from this group of boreholes to maintain high CH$_4$ concentration.

E. Drill extended length bores into the A Seam from the B seam tailgate road. The A seam, located 20-25 metres above the B seam, is approximately 3.0 metres thick and is a significant source of gas emission into the B seam longwall goaf. The purpose of these borehole is to extract gas released from the A seam as it is fractured, thereby reducing the volume of gas liberated into the B seam goaf.

F. Drill a regular pattern of pre-drainage bores across the adjacent longwall block and gateroad. The purpose of these bores is to pre-drain the B seam and reduce the gas content prior to mining.
In addition to the improvement actions listed above, a second round of investigation and onsite testing is recommended to identify and evaluate additional actions that may further improve the effectiveness of the mine ventilation and gas management systems and support increased productivity and reduced operating cost. Methods to stimulate increased gas production from low permeability and undersaturated coals seams, such as cyclic inert gas injection (Black 2011 and Black 2013), should be considered for site testing and evaluation.

CONCLUSIONS

This project aimed to identify and evaluate practical improvements to reduce the impact of CH$_4$ on longwall productivity. Ventilation and gas emission modeling confirmed that modifying the ventilation circuit to include an additional heading between the installation face and main return (MG Bleeder Road) and directing the bulk of longwall return air flow through this bleeder road has the potential to support a 100% increase in longwall production. Modeling also confirmed that pre-draining gas from the B seam and increasing goaf gas extraction offers a minimum 100% increase in longwall production.

To support further increases in longwall production it will be necessary to (a) reduce losses and inefficiency in the mine ventilation system to increase the quantity of ventilation air available to the longwall panel, (b) implement pre-drainage of the working seam and adjacent coal seams, (c) expand the goaf drainage program to include an increased number of boreholes, and (d) expand the drainage reticulation system to efficiently extract increased gas volumes.

Longer term actions that offer a step change improvement in development and longwall productivity include:

- Introduce an alternate roof support system, such as roof bolts and cable tendons, to replace steel arches;
- Increase the cross-sectional area of the airways in the longwall ventilation circuit to reduce air velocity which presently prevents ventilation air quantity being increased to dilute CH$_4$ emissions; and
- Develop two (2) heading gateroads to replace single entry.

REFERENCES


MODERNISING AN UNDERGROUND GAS DRAINAGE SYSTEM IN RESPONSE TO INCREASED PRODUCTION AND GAS CONTENT

Andrew McInerney¹ and Miles Brown²

ABSTRACT: This paper highlights a successful step change in the management of increasing gas contents and compressed drainage timeframes. The improvements have overcome safety risks concerned with a system not suitable for handling the required gas loading for current and future production targets. The paper discusses how the upgrades manage increasing seam gas content, high rig drilling rates, record development and longwall performance. Improvements to the system included specific design to suit continuity, in-seam hole stability, predicted peak gas flows, drilling and drainage direction, infrastructure type and capacity, formation of empirical gas decay curves and fines and water removal from the system. Using data captured from the commencement of the upgrade, steps toward an efficient and malleable long and short term design and planning tool have been taken. Variable flow rates has led to the investigation of high fluctuation of gas capture, moving away from traditional prediction methods and relying on analysed mine specific data.

INTRODUCTION

Anglo-American’s Grasstree Underground Mine operates the German Creek Seam in the Bowen Basin. Production for 2015 is approaching 10Mt. Underground in-seam gas drainage with only two in-seam drilling rigs is utilised to drain the German Creek seam. With virgin gas contents approaching 16m3/t in the German Creek Seam and with Methane as the predominant gas, the management task is large. Increased production with increasing gas content is the real challenge for Grasstree’s underground in-seam gas drainage management team. In April 2015, following incidents involving gas emissions at drill stubs and development faces, the complete system needed an overhaul. This paper quantifies all the changes that have been introduced and discusses the intense reconciliation that occurs to ensure all changes are successful. The system will only be deemed successful if safety concerns are mitigated and predicted values are achieved. Hence by managing safety as the primary objective for changes to gas drainage, the technical improvements at the site have followed. Additionally any changes to the Underground In-seam (UIS) drainage system must cater for ever changing geological situations. The drainage designs and drainage infrastructure used require flexibility within their designs to suit all situations that are foreseen. Permeability, dykes, faults, gas content all vary in size and eliminate the ability of one design fits all.

APPROACH FOR UIS GAS DRAINAGE

To achieve change there must be a solid framework, which provides a clear flow path to success. This framework needs to outline all principles that provide the solution. The two over riding key governances for ensuring that gas drainage is effective at Grasstree Mine are;

- Science – where gas drainage rates, gas flows, gas pressures and business continuity values are predictable
- Design – where all potential safety, geological and structural risks are engineered to as low as possible

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² Director Drive Mining Pty Ltd, 0458768620, E-mail: drivemining@bigpond.com
The Grasstree underground gas drainage upgrade was aimed at creating a “Gas Drainage System” that is transparent in all its designs and all its predictions. The system is best shown in Figure 1.

The key principles to be continually used for an effective scientific approach are:

- Prediction
- Continuity
- Monitoring

Prediction

Utilising first principles to form an operational design program requires understanding on how drilled drainage holes actually react. This can be done with or without permeability data. Analysing hole flow data from existing underground in-seam holes, where known virgin gas contents is the primary method. This data along with monitoring the hole right through to the compliance core result can create a base decay curve. Once the decay curve is developed, and for Grasstree there are two distinct curves used for predicting flows, the other principles are applied to estimate hole flows.

Principles for hole flow estimation and subsequent drainage decay include;
1. Drilling rate – metres/day or metres per week
2. Hole spacing – this is the variable when determining continuity requirements
3. Seam thickness
4. Virgin Gas Content and target Gas Content
5. Hole length
6. Decay Curve (Figure 2)
The output from the design program is estimated individual hole gas flows and total site gas flow profile. (Figure 3). This information allows for future tracking of hole performance.

Figure 3: Predicted gas flow

Continuity

Gas Drainage continuity with development is achieved using the Grasstree gas drainage design program in conjunction with the mine production profile. The system is used for both single holes and drill sites and is also used for Life Of Mine (LOM) continuity. Life of Mine planning becomes a simple task of ensuring the correct spacing is applied to achieve continuity. The following two figures represent the planning spreadsheet tool for an individual drill site.

Figure 4: Planning spreadsheet A

Drainage continuity with development is achieved by modifying the drainage hole design to suit the variable permeability, quantified by specific gas decay curves, to set a hole spacing that achieves a planned gas drainage target content of approximately 3 m$^3$/t.
### Figure 5: Planning spreadsheet B

<table>
<thead>
<tr>
<th>Drill Site</th>
<th>Target Zone</th>
<th>Area to Drain (m²)</th>
<th>Ave Seam Thickness (m)</th>
<th>Tonnage Coal to Drain (t)</th>
<th>Vol of gas to remove (m³/t)</th>
<th>Start date of Gas Flow to pipe</th>
<th>Expected Time to drain to target (Days)</th>
<th>Required Finish date to drain to target</th>
<th>Continuity with Development at commencement of Zone</th>
<th>Initial PERK total flow from holes (total)</th>
<th>Total flow from holes at 3m³/t</th>
<th>Probable Gas Content when Mined (m³/t)</th>
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<tbody>
<tr>
<td>Hole 1 - Inbye</td>
<td>13 to 13.5</td>
<td>24700</td>
<td>2.7</td>
<td>93366</td>
<td>1027028</td>
<td>28/06/2015</td>
<td>134</td>
<td>16/12/2015</td>
<td>9/11/2015</td>
<td>411</td>
<td>17</td>
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<td>12.5 to 13</td>
<td>17700</td>
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<td>66906</td>
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<td>Hole 3</td>
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<td>1056132</td>
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<td>19/11/2015</td>
<td>1/12/2015</td>
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<td>12/07/2015</td>
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<td>6/11/2015</td>
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<td>Hole 7 - Outbye</td>
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<td>27/12/2015</td>
<td>79</td>
<td>372</td>
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</table>

### Figure 6: UIS Design Drainage time to 5m³/t and 3m³/t

### Figure 7: Life of Mine Planning for Drainage hole spacing
Monitoring

The key component to any gas drainage system is acquiring reliable data. Every UIS hole is fitted with a gas flow measuring set outbye of the holes isolation valve. Two styles have been utilised. High flowing holes (>30 l/sec) utilise orifice plate styles. This is so that when the hole is not being measured the inserted orifice plate is removed to stop potential blockages. The Venturi style is employed as required on lower flow holes or lower risk holes. The orifice plate style is the predominant tool. This is because there is less chance of a blockage especially as Grasstree UIS holes release a high volume of material.

Weekly individual hole flow monitoring is in place. This data is placed into operating spreadsheets for determining how the hole is draining against predicted flows. Anomalies identified from these actual vs. predicted flows then allow the site to remedy low flowing holes or analyse high flowing holes. Additionally, holes which are found to have a blockage can then be treated. Furthermore, every surface riser has real time monitoring allowing for accurate calibration of decay curves and reconciliation of results.

Figure 8: Hole Spacing calculator

<table>
<thead>
<tr>
<th>UIS Hole Spacing m/s</th>
<th>16</th>
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<th>14</th>
<th>13</th>
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<td>73</td>
<td>70</td>
<td>67</td>
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<td>57</td>
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</tbody>
</table>

Figure 9: Single hole standpipe arrangement with orifice plate
DESIGN – UNDERGROUND GAS DRAINAGE

The key principles to be continually used for an effective design approach are;

- UIS Hole Design
- Infrastructure

UIS hole design

The following diagram depicts the principle design being utilised for remaining LOM UIS drilling. The design variable is the hole spacing at the target future development gate road. Spacing will vary with regards to continuity with development (schedule), gas content variations and lower permeability zones (identified from geological interpretation and micro cleat analysis).

The new UIS design has numerous strengths. These include;

1. UIS hole stability improvement. Previously holes would fail during or after the hole was completed. The majority of these issues occurred on cleat direction where the drill bit struggled to maintain the desired drilling direction. The holes also failed where branches were occurring on the same direction as the cleat. The new design requires branching to avoid cleat directions at all times. The design principle is - “all branches are to be directed away from cleat angles”. Branches are planned and sequenced to ensure that this occurs. Since the change there have been minimal issues in hole and after completion.

2. Hole branching. Hole density has increased at the target locations where development are to mine. This is achieved by reducing standpipes and increasing planned branches. This design has an added control that the last branch drilled from any standpipe is the first hole intersected by development. This is to reduce chances of long hole blockages.

3. Hole direction. The design has identified an improved drainage direction when drilling through planned gateroads or zones of very low permeability and drilling difficulty. The additional benefit is that when development intersect these holes in the gate road the virgin side of the hole is in

![UIS design principles diagram](image-url)
the rib line while the suction side is in the face. This naturally improves borehole management when developing. Additionally, direction when drilling through geological structures (dykes/fault zones) allows enhanced stability and drainage when the correct angle is chosen.

2. Directional permeability is evident at Grasstree and has been backed by recent micro cleat analysis at UNSW in these zones. Micro cleat analysis validates this assumption as areas of good drainage show a micro cleat clear of obstructions. The new design has rotated the angle of the holes so as to not be parallel with lower permeability directions.

3. Roof touches. All holes require a roof touch prior to drilling across a planned gate roads. This is to achieve highest possibility of the hole being mid seam in these areas, maximising gas drainage. Mid seam intersections on development are far easier to manage.

4. End of UIS hole sump. All UIS holes and branches now have a sump at the end of the holes to accommodate drill fines and reduce chances of holes being blocked upon intersection in development. These sumps are 12 m in length and are drilled down to the floor. Tails are 45 m past the gate road to suit these tails. Additionally these sumps allow flanking drill holes to avoid the end of the holes, reducing the chance of interaction.

5. Virgin gas content cores. At least one core is taken when drilling a full pattern. In conjunction with initial hole flows, an accurate virgin content allows for immediate recalibration of the prediction model. This data is also fed into the “NEW” planning spreadsheet to better plan for how a section is going to be drained.

6. Infrastructure size. Predicting flows is an output of the planning spreadsheet and determines the pipeline and gas riser size. Infrastructure (pipes) needs to suit predicted flows. A single UIS site has recorded flows peaking up to 2500 l/sec which needs to be managed by infrastructure or planning.

7. Fines management. In-hole fines creation occurs during the drainage of the German Creek seam. These fines and stone fragments are predominantly emitted into the drill site gas pipework, lifting and removing it when the gas pressure and flow is at it’s peak. It is paramount for the infrastructure design to have the ability to remove this waste underground through a water trap off the range. The remaining fines, so as to not cause a build up or blockage in-hole, are targeted at being deposited in the end of hole sump. The final 12m’s of the hole is drilled downwards to create this sump.

8. Structure identification. The UIS design has the ability to be changed where there is an identified or a predicted outburst prone structure(s). “Close the grid” style drilling is used to attempt to locate these structures. It should be noted that small structures are very difficult to predict with UIS drilling as they can be penetrated without noticing changes. These “close the grid” designs can be conducted at the commencement of drilling by “looping” the tails of the holes across each other or just prior to mining by drilling from behind the development operations. The earlier a structure is identified means a lesser chance of a structure being unexpectedly intersected by development. Once structures are identified then additional compliance cores can be designed and conducted prior to development. Figure 11.
9. Standpipe placement. Holes are drilled left to right in a drill site to maximise room for driller operators. Standpipes are limited by the hole spacing. Basically two branches off the trunk are designed for every hole that targets a future gate road. Figure 12.

Figure 12: UIS design change

Infrastructure

The current drill site arrangements can relate to both stubs or open sites. The drill site design standard has the objectives of both allowing operators a less constricted site (ergonomical) and provide a separate water and fines system allowing flow to the gas riser. The arrangement requires flexibility depending on the location of the riser. Each site will be arranged to suit, however the general layout applies. There must be:

1. A method of isolating water and fines from the riser and pipes in order to remove from the system. (see water trap in Figure 13).
2. Equipment installed for measuring individual hole gas flows, known as Measuring Sets.
3. Infrastructure in line for allowing holes to be unblocked without releasing gas to the atmosphere. This is by a 100 mm (4") to 50 mm (2") t-piece between the standpipe and hole isolation valve.
4. Adequate pipe infrastructure to allow gas to flow to the gas riser with minimal restrictions
5. Pressure monitoring in pipe infrastructure to ensure that the pressure TARP is easily managed. Installing the correct size riser assists with reducing the chances of high pressures. The decision tool for the riser diameter is shown in Figure 14
6. Adequate height differential for separating driller gas and water/fines to their fines bins. This will minimise water and fines inflow into main pipe range from the drillers.

Figure 13: Drill site pipe infrastructure
RECONCILIATION OF DESIGN

Predicting and monitoring of gas flows and mining gas content

The main output of the prediction and monitoring model is the final gas reconciliation for purposes of guaranteeing precise gas capture from the reservoir, model accuracy and correction and low gas content upon coring and mining. Figure 15.

Case Study- 905MG 9ct A Heading

This is the second new style drill pattern to be intersected by development. All core results were below 3 m³/t and suction was seen at face upon intersection by development for all holes. Figure 16, shows a comparison between actual measured flow rates versus the predicted flow rates for the pattern shown in Figure 19. This actual flow data gathered is used in a process of reconciliation of the individual borehole reservoir to predict residual gas content at desired times to develop robust compliance core schedules to ensure development continuity. This flow data and characteristics is fed back into the decay model by
means of comparing gas captured (volume) to the flow rate of the hole at a particular time from commissioning. This makes the prediction of flow rates at a set gas content more accurate for future patterns in similarly permeable areas.

Figure 16: UIS design predicted vs. actual gas make

Figure 17: 905MG 9ct actual drilling and compliance results
Figure 18: 905MG 9ct actual individual hole flows

Figure 19: 905MG 9ct actual individual hole flows
Figure 20: UIS reconciliation tool

ADDITIONAL IMPROVEMENTS

Microcleat analysis

Although it has been identified through monitoring that there are definite zones of lower permeability (lower gas flow) the reason has not been proven. Dr Lila Gurba from the University of New South Wales was engaged to analyse coal samples for potential flaws to coal gas flow. These tests also looked at areas where there were poor gas drainage flows and good gas drainage flows encountered. These tests were at a micro level.

The analysis has shown a definite directional issue with micro cleats thwarting gas migration in one direction. The direction is clear and appears to be sheared closed hence the poor gas flow. More analysis is to continue.

Extended Q1 analysis

Grasstree mine utilises both surface and underground coring for both compliance gas content cores and virgin gas content core data. To take a core from underground at a distance which would normally take greater than 40 minutes to place under test, the site required a method to be acceptable to allow this to occur. GeoGAS was engaged to provide a correction factor for this purpose.

The following diagram represents Grasstree Q1 correction factor for when cores take greater than 40 minutes to be put onto test. The advantage of this test increases the use of longer cores or cores where there were issues recovering, providing data well before current time limits allow.

Figure 21: UIS design change

Borehole intersection suction level TARP

Creating a TARP for suction levels prior to development intersection is the final key to the puzzle for improving mine safety for UIS drainage vs development interaction. Improvement and quality standards of roadway hose over standpipes and methods is also vital to successful gas control post intersection.
CONCLUSIONS

The development of a reconcilable underground gas drainage system is the key to sustaining effective gas drainage for the remainder of the mines life. This system must and can cover all changes in coal characteristics in relation to varying gas content and effects from geological structures. The following points highlight the success of this system.

- Underground Inseam hole gas flow is able to be estimated accurately with or without permeability data. Variable decay curves can be created and calibrated with regards to different coal characteristics
- Understanding microstructure is vitally important to understanding hole flow variations. Also a link with microncleat issues and outburst prone structures or even coalburst characteristics
- Correct Infrastructure design (size) is required to limit reduce gas pressure increases from hole flows or from gas surging. This includes gas pipe or gas riser diameter
- Being able to reconcile the complete design of a gas drainage system in regards to its performance versus planned is vital for not just approval to mine but for the workforce confidence, especially for such a gassy operation.
- Designing a system that sets standards for suction levels required to be applied to UIS holes prior to developing thru is a massive step in reducing potential for gas incidents in development faces.

The final hurdle is the opinion of the crews and staff at site. The support for change was always positive, however the results and confidence gained for the current management of gas drainage is justified. Providing a new handbook for all aspects of Underground gas drainage now allows all on site to understand the volume of processes conducted at Grasstree. This handbook will be used for all training aspects of gas Drainage and as a support document to Principle Hazard Management Plans.

ACKNOWLEDGEMENTS

The opportunity and financial support provided by AngloAmerican and the Grasstree Mine to undertake this work is greatly appreciated. The assistance and input provided by Drive Mining Pty Ltd and in particular Miles Brown is also acknowledged.

Dr Lila Gurba from the UNSW has provided analysis of coal properties at the microstructure level and Geoff Williams from GeoGAS for completing the extended Q1 Analysis

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COALBURST CAUSES AND MECHANISMS

Justine Calleja¹ and Jan Nemcik²

ABSTRACT: Coalburst (also known as coal bump) is a well known phenomenon in underground coal mines internationally, however, it was not recognised as a risk for Australian coal mines until the recent double fatality at Austar Coal Mine in the Hunter Valley in 2014. This paper reviews the international knowledge base from research and practice to provide Australian mining professionals with an understanding of the basics of coalburst causes and mechanisms in order to allow mine operators to address the risk of coalburst in mining safety management plans. This is the first of two companion papers with the second paper, “Coalburst control methods” (Calleja and Porter 2016).

INTRODUCTION

There has only been one published case of coalburst in Australia – the Austar fatalities reported in 2014 (NSW Mine Safety Investigation Unit 2015). The investigation report recommended that, when encountering pressure burst (coalburst) conditions, mine operators should consider developing a management plan which takes into account a complete worldwide literature search of publications relating to pressure bursts. An extensive international literature search and review has been completed, and this paper summarises the key publications and international experience on coalburst which have been reviewed. This paper can be used as a reference source in combination with any other similarly high quality source to meet the investigation recommendations. It should be noted that any publications (including this one) should be assessed critically based on the quality of real world evidence provided to support findings, and with advice from suitably qualified and experienced professionals prior to engineering application.

SIGNIFICANCE AND OCCURRENCE OF COALBURST

Whilst the Austar fatalities are the only published case of coalburst in Australia, there has been a long recognition of the occurrence of outburst (bursting of coal due to high gas pressure) in Australia (Hargraves 1980). As a result of the recent identification of coalburst as a separate phenomena from outburst, many researchers and operators in the coal industry are now questioning whether many of the outbursts observed previously may have actually been caused by stress rather than gas pressure or were caused by a gas - stress combination mechanism and were incorrectly labelled as outbursts. Irrespective of whether this turns out to be true, it is certainly true that coalbursts have been occurring in Australia for many years at Austar Mine (NSW Mine Safety Investigation Unit 2015) and similarly there are likely to be many unpublished cases of coalbursts in other Australian mines. However, evidence of these cases is yet to be published and the extent of the risk of coalburst in Australia is still to be defined. This work will form an important component of future research efforts to define and manage the risk of seismicity in underground coal mines in Australia.

Potvin and Wesselloo (2013) state the seriousness of the risk of mining seismicity in unequivocal terms, “The possibility of experiencing a seismic event resulting in fatalities has arguably become the most important financial risk in underground hardrock mines operating in developed countries. In the two most recent cases in Australia, the entire operation was shut down for a period well exceeding one year while the mining method had to be re-engineered in order to demonstrate to regulators that the seismic risks had been lowered to an acceptable level.”

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The development of seismic risk is often progressive, as a result of increasing stress and seismicity as mining progresses to increased depths. Lack of recognition and analysis of seismic risk in mine development and planning has led to inappropriate mine designs and inaccurate assumptions around percentage extraction and mining rates, which have led to mines performing at lower profitability, the occurrence of injuries and fatalities and mine closures (Beck and Duplancic 2005). Mining seismicity has been occurring and has caused injuries and fatalities in Australian hard rock mines since the first reported event in 1917 in Kalgoorlie. Mark and Gauna (2015) noted that major bursts in the USA have often been preceded by a pattern of increasing coalburst activity and Whyatt (2008) found that for clusters of three or more bursts in a 12 month period since 1999, one ended with mine closure, two continued without incident, two resulted in a design change or move to a new mining area, one ended with a fatality and one ended with a fire and explosion (Figure 1). It should be noted that Crandall Canyon was on this graph with two coalbursts in the 12 month period, but which later had a pillar burst disaster in August 2007 which caused nine fatalities.

Figure 1: Reportable coalburst grouped into clusters by mine and 12 month period through to July of 2007 in the USA (Whyatt 2008)

Table 1 shows the history and occurrence of coalburst internationally. Earliest experiences of coalburst occurred in Europe as a result of long term coal mining and the development of very deep, often multi-seam, coal mines. A great deal of the current understanding of coalburst and its management evolved in Europe and has since been adopted and extended with advancing technology in China and Europe. Whilst 927 fatalities have been reported in time periods which are not overlapping in the references listed, this is by no means a complete record. It is likely that the actual number of fatalities caused by coalbursts internationally is significantly greater.

A number of terms have been used in different countries to describe the same phenomenon, which is referred to as coalburst in this paper. In China, Europe, South Africa, Russia and India, coalburst is described as rockburst of coal and this is the most technically accurate terminology. For the purposes of clarity in this paper, ‘rockburst’ will be used to describe bursting of non-coal rock and ‘coalburst’ will be used to describe bursting of coal. In quotations, where a different word is used to mean coalburst that word will be superscripted with ‘cb’ e.g. rockburst\(^{cb}\). Coalburst is sometimes described as ‘coal burst’ (with a space) internationally, however in Australia, the terms ‘rockburst’ and ‘outburst’ are both written without a space and so this practice is also adopted here for coalburst. The singular form of the word ‘coalburst’ is used as a noun to describe the phenomenon or a single event. The plural, ‘coalbursts’, is used to describe more than one event.

Many researchers, globally, have defined rockburst and coalburst and there is strong agreement in their descriptions of the phenomena. Kaiser and Ming (2012) defined a rockburst as “damage to an
excavation that occurs in a sudden or violent manner and is associated with a seismic event”. Brauner, 1994, wrote that “Every rockburst is accompanied by a loud report and ground tremor – a seismic shock. The rockbursts in coal mines are violent failures of the coal seam, causing ejection of broken coal and often taking the form of an abrupt movement of the face or sidewall.”. MSHA (2004) provided the most specific definition of rockburst as a “sudden and violent failure of overstressed rock resulting in the release of large amounts of accumulated energy. Rock burst does not include a burst resulting from pressurized mine gasses.”

There are two essential defining factors which allow an event to be categorised as a coalburst. Firstly, there must be sudden and violent ejection of coal. Secondly, the coal failure must be associated with (the cause of, or caused by) a seismic event.

A seismic event is a sudden inelastic deformation within a given volume of rock, i.e. a seismic source that radiates detectable seismic waves. It is defined quantitatively by the seismic moment, M, and either radiated seismic energy, E, or stress drop, Δσ (Mendecki et al., 1999). A seismic wave is an elastic strain wave which propagates through rock. A simple analogy is that of a ruler, which is clamped on a desk at one end. The other end is free to bend. As the free end is bent further from horizontal, the ruler is straining elastically and storing strain energy. When the ruler breaks, the strain energy is released and the free end of the ruler will vibrate up and down. The clamped end of the ruler cannot move freely, but as the free section of the ruler vibrates it alternately compresses and tenses against the desk, which changes the state of stress and strain in the desk around the ruler in a reverberating manner thus creating a seismic wave in the desk.

### Table 1: Coalburst occurrence and fatalities by country / region

<table>
<thead>
<tr>
<th>Country / Region</th>
<th>Earliest known coalburst</th>
<th>Time Period</th>
<th>Number of Coalbursts</th>
<th>Number of Fatalities</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Czech Republic / Poland</td>
<td>1880</td>
<td>1983-2003</td>
<td>190</td>
<td>122</td>
<td>(Ortlepp, 2005)</td>
</tr>
<tr>
<td>Ruhr, Germany</td>
<td>1890s</td>
<td>1973-1992</td>
<td>50</td>
<td>27</td>
<td>(Brauner, 1994)</td>
</tr>
<tr>
<td>USA</td>
<td>1924</td>
<td>1943-2003</td>
<td>78</td>
<td></td>
<td>(Blake and Hedley, 2003)</td>
</tr>
<tr>
<td>USA</td>
<td>1924</td>
<td>1983-2013</td>
<td>337</td>
<td>20</td>
<td>(Iannacchione and Tadolini, 2015)</td>
</tr>
<tr>
<td>China</td>
<td>1933</td>
<td>1933-1996</td>
<td>4000</td>
<td>400</td>
<td>(Zhou and Xian, 1998)</td>
</tr>
<tr>
<td>China</td>
<td>1933</td>
<td>2006-2013</td>
<td>&gt;35</td>
<td>&gt;300</td>
<td>(Jiang et al. 2014)</td>
</tr>
</tbody>
</table>

**WHAT IS COALBURST?**

Any type of rock failure will release some seismic energy, however the amount of seismic energy released by a coalburst will be significantly greater than the seismic energy released by non-violent failure of coal.

Coalburst is distinguished from progressive stress induced failure or gravitational failure by the release of stored elastic strain energy in the form of both seismic energy and kinetic energy in the process, which will propel the failed coal a greater distance from its original position, than could occur as a result of gravity alone (Figure 2).
Coalburst can occur irrespective of gas content. In cases where the in-situ gas content is low, a coalburst will not be associated with a large release of gas into the workings and can be recognised by the monitored or observed seismic event and the distribution of failed coal during or after the incident.

Gas outbursts always occur with a large release of gas into the workings, and the failure of coal from gas pressure will impart kinetic energy and propel the coal into the workings. Ortlepp, 2005, and Brauner, 1994, agree in their definitions that outbursts are events which are generally not regarded as coalbursts and merit a separate classification as very little seismic energy is emitted into the rockmass. In contrast, a number of researchers have considered gas outbursts to be a sub-set of coalburst, whereby the coal failure is instigated by stress and the coal propagation is supported by released gas pressure (Lama and Saghafi 2002). In China, Li et al., (2007) suggested that coalbursts can sometimes lead to outbursts, rather than outburst being a separate non-seismic or independent phenomenon. In addition, Li et al., (2007) wrote, “In coalmines the correlation of gas outbursts and rockbursts is very strong, especially at depth. A different type of rockburst, or gas outburst-rockburst can be triggered by the coupling effect of unloading of confining stress due to mining and desorption and expansion of high-pressure gases. In addition high-pressure gases can contribute to rockburst. As a result the rockburst and abnormal gas gush can be used as warning signals interchangeably.”

CAUSES

There are many examples and case studies of coalbursts which demonstrate that they can occur in a range of different circumstances with different causative factors. Coalbursts have occurred out of the face of development headings, as rib bursts out of a development pillar or on the block side of the roadway, out of the longwall face, out of the longwall block or out of the pillar side ribs in longwall gateroads. In addition, outbye pillar ribs and block ribs have burst as well as entire pillars and areas of pillars (pillar burst).

Coalbursts are often triggered by the mining process and usually occur close to an active mining face in development, longwall or pillar mining but can also be triggered by blasting or large scale mining-induced seismic events (e.g. magnitude 1-3). Brady and Brown (1994) defined two broad classes of rockbursts. Type 1 results from fault slip events and Type 2 results from failure of the overstressed rockmass. ‘Shakedown’ can also occur as a result of mining induced seismicity, however this is a result of already fractured rock or coal collapsing due to shaking and gravity when it is not sufficiently contained by the installed support. Shakedown is an important risk associated with mining seismicity but it is not rockburst or coalburst as the rock failure mechanism is not dynamic (Whyatt 2008).

Iannacchione and Zelanko (1995) reviewed the MSHA coalburst database and found that “pillar retreat mining accounted for 35% of the bumps, barrier-splitting for 26%, longwall mining for 25%, and
development mining for 14%. Of the longwall incidents, 33% affected the longwall face, 19% the tailgate entries, 36% both the longwall face and the tailgate entries, and 6% the headgate entries”.

Brauner (1994) provided illustrative cases of coalbursts which occurred in the Ruhr, Germany. In one case (1000 m depth with a 40 m thick sandstone roof unit) a pre-existing roadway in the longwall block and more than 100 m of the longwall face burst as a result of increasing vertical stress due to the diminishing pillar created as the longwall face approached the roadway. In another case, at 720 m depth and with a 25 m thick sandstone roof, a 35 m length of rib of a roadway adjacent to mined workings in an overlying seam, 57 m above, burst more than 4 months after the longwall face had approached to within 22 m. Another case occurred on the tailgate end of a longwall face at 800 m depth. The tailgate pillar had a yield design (2-7 m wide) and there were thick units of sandstone in the roof and floor (24 m and 28 m). 15 m of the longwall face burst. One miner was killed and nine were injured. After the burst a gap of 15 cm between the seam and roof was evident. The roof and floor remained stable.

Brauner (1994) identified three critical contributing factors which all need to be present for coalburst to occur, they were: high static stress, triaxial loading of the coal (i.e. presence of sufficient confinement) and the presence of a strong thick and massive lithological unit in proximity to the seam. Many researchers have recognised the importance of the combination of mining conditions and geological features in the causation of coalburst (Whyatt et al., 2002). More recently, Mark and Gauna (2015) reviewed the risk factors for coalburst in the USA and internationally. The key factors identified through these works to indicate coalburst risk were:

**High static stress**

The level of static stress required to cause coalburst is proposed by Brauner (1994) to probably be in the order of 100 MPa and with the coal subject to triaxial loading conditions. This may occur due to high depth, proximity to goaf and particularly tailgate corners, overlying unmined areas adjacent to workings and/or critical pillar dimensions (width to height ratio between 5 and 20).

- Depth of cover is an important contributor to static stress. Whilst coalburst has occurred at 230 m depth in the USA, and Brauner, 1994, reports a case at 100 m depth, the number of coalburst incidents increases dramatically as depth increases. Almost half of the mines operating over 600 m experience coalburst.
- Pillar design plays an important role in concentrating vertical stress. Critical pillars with a width to height ratio greater than 4 - 5 and less than fifteen, have been identified as high risk, as they are too large to fail under low loads in a static way (as yield pillars do) but create higher stress concentrations than larger abutment pillars (Iannacchione and Tadolini 2015). Brauner (1994) shows cases of coalbursts with width to height ratios between five and twenty, however, Brauner’s work and other international evidence indicates that there is no safe pillar size above the yield pillar range which will not burst (e.g. longwall blocks). Yield pillars are designed to fail prior to longwall abutment loading and redistribute stress away from the critical maingate and tailgate roadways. However, this has, at times, led to high vertical stresses being concentrated on the longwall block and longwall face leading to coalbursts.
- On a larger scale, mine layout is important in creating vertical abutment loading. More than 80% of bursts have occurred on retreat longwall or pillar mining, and only 20% have occurred on development. Modifying the mine layout to avoid creating corners surrounded by goaf or infrequent goaf failure can be used to reduce stress concentrations. Avoiding cutting into highly stressed pillar cores in pillar mining and reducing mining rates to allow progressive failure around the excavation reduce the risk of coalburst.
- Remnant coal left after other seams have been mined will create stress concentrations which have been a factor in many cases of coalburst in the USA and internationally. The distance between the previously mined seam and current workings, as well as the geometry of previous workings has a significant effect on the level of risk. When more than one seam has been previously mined and remnants overlap particularly high stress concentrations can develop.
Structures, such as faults, can also act to prevent stress transfer and distribution, allowing high static stresses to be concentrated around them, either from a tectonic or mining induced origin.

Rapid changes in depth of cover have been associated with coalbursts, however the exact role they play has not been determined. It may be due to stress concentration under incised valleys, otherwise undetected structural features which resulted in the development of steep topography, increased vertical stress from increased depth of cover or some other unidentified cause.

Lithology

- The presence of strong, thick and rigid strata more than 5 m thick and close to the seam (within 10 m) is an important risk factor. Brauner (1994) wrote that, “Burst tendency decreases or vanishes as soon as the main roof gives way”. The identification of high risk strata can be achieved by exploration or underground borehole geotechnical characterisation using coal mine roof rating unit ratings combined with rock testing results (Calleja 2006). High risk units would be expected to have a unit rating of 50 or more (Calleja 2008).
- Sandstone channels of 1.5 m within 1.3 m of the seam have been found to cause coalbursts (Hoelle 2009). Sandstone channels are also known to occur at Austar (NSW Mine Safety Investigation Unit, 2015). Seam rolls are an additional feature which have been associated with coalburst.
- Seam thickness – 4 m - 6 m thick seams appear to be the most burst prone, although Brauner (1994) suggested that this was due to the mining methods used in those cases. An increased risk of coalbursts in 4 - 6 m thick seams has been observed in China, and Austar is an Australian mine with coalburst which supports this as a risk factor.
- Coal properties – many different researchers have found that almost any coal can burst, and coal strength and other properties cannot be used to rule out coalburst risk. In saying that, the elastic properties of coal are measured in the Czech Republic coal mines to indicate coalburst propensity (Ptacek 2015).

Dynamic loading and Seismic Events

A number of coalbursts have happened at the time of shotfiring at a distance within 30 m or more. There are mines which have numerous mining-induced seismic events but are free from coalburst, however coalburst prone mines may have coalbursts triggered by external seismic events. Faults which can be unlocked by local or regional mining induced stress changes can slip and cause seismic events which can trigger coalbursts.

History of coalburst

Most of the worst coalburst incidents were preceded by less severe coalburst events. As such a history of coalbursts within the seam being mined is considered a moderate risk for coalburst, and a history of coalbursts at the mine is considered a high risk.

One case study from the USA is particularly interesting, from a longwall mine in Eastern Kentucky (Hoelle 2009). Coalbursts were experienced at a minimum depth of 358 m and with many occurring in the 400-600 m depth range. Major coalbursts occurred in the tailgate abutment pillars and the tailgate end of the longwall face with seismic events of magnitude 2 to 4.3. Initially, the presence of sandstone channels (1.5 m thick) at the top of the seam were identified as causative. However coalbursts also occurred where the sandstone channels were not present, but the immediate roof consisted of sandstone or siltstone with UCS of 100 – 177 MPa and Young’s Modulus of 20-33 GPa.

MECHANISMS

Rockburst has traditionally been described as being the combined action of shearing and subsequent splitting resulting in sudden detachment of rock slabs with a high velocity and is usually observed in
brittle hard rocks (such as unweathered igneous or siliceous sedimentary rocks), which have a sudden loss of strength following little or no plastic deformation (Vutukuri and Katsuyama 1994). Brady and Brown (1994) provided a more detailed analysis of the failure process of rockburst, defined as “the release and transmission of seismic energy from the zone of influence of mining. It is well known that impulsive loading of a structure member results in transient stresses greater than the final, static stresses in the system rockbursts may be best studied through methods which account for energy changes in the system”. Rockburst is known to occur as a result of differences in the post-peak stiffness of the rock and the loading environment as a result of the accidental bursting of rock samples in the laboratory before the advent of servo-controlled testing machines.

In the laboratory, rocks, such as coal, which show strain-softening behaviour (brittle rock) after failure under uniaxial loading continue to deform with rapidly decreasing load. If the loading environment is softer (less steep gradient on the stress / strain curve) than the specimen, then the strain energy released by the loading environment is greater than strain energy which can be absorbed by the rock. The excess energy is converted to seismic and kinetic energy and results in bursting. Where $\Delta W_m > \Delta W_s$ in Figure 3 a) the excess energy results in rockburst.

This analytical approach to understanding sample bursting in the laboratory can be extended to a larger scale, where, if the stiffness of surrounding rock which is loading a pillar or ribline is lower than the post-peak stiffness of the pillar or the ribline, then it will burst. Specifically, rockburst or coalburst will occur if the surrounding rock is able to deform and continue to apply higher loads than the failing rock can absorb.

**Figure 3: Post Peak unloading using machines that are a) soft, and b) stiff, with respect to the specimen (Brady and Brown 1994)**

Based on this conceptual approach, the key factors required for rockburst are that:

1) The rock is in an intact state prior to failure, in order for there to be stored elastic energy which can be released,
2) Has been sufficiently stressed to exceed its peak strength,
3) Its post-peak behaviour is strain-softening, and
4) It continues to be loaded by the surrounding rock with a higher load than it can absorb as it fails.


The majority of coalburst cases reviewed by Iannacchione and Tadolini (2015) were due to the excessive stress mechanism, which typically occurred at depths greater than 500 m in retreating partial or full extraction mining, close to a pillar line, when advancing beneath overlying remnant pillars or at a mining face which had not been successfully de-stressed. The excessive stress mechanism occurs when coal, which was stable on development is exposed to rapidly increasing vertical stress. This failure mechanism is reasonably explained by the laboratory failure mechanism in Brady and Brown (1994).
The Crandall Canyon disaster in 2007 is a widely known, well documented example of this mechanism (Figure 2).

The occurrence of both rockburst and coalburst events as a result of large scale mining induced seismicity has been well documented. Brady and Brown (1994) discussed the importance of dynamic loading in the following terms, “it is well known that impulsive loading of a structural member results in transient stresses greater than the final, static stresses in the system...the amount of energy that a particular member can store or dissipate is frequently an important criterion in mechanical design”. However, understanding the role that dynamic loading and seismic events have on excavations requires more than just analysis of impulsive loading, and it is not explained by the post-failure stiffness model. It requires consideration of the transmission of seismic waves and energy in rock.

The fundamental issues which can explain seismic triggered rockburst and coalburst rely on defining the impact of seismic waves in the excavation boundary and the rapid stress changes and deformations they produce. It has been hypothesised that seismic shock bursts can be triggered by the shear failure of rigid strata during goafing and subsidence Figure 4) and by the sudden failure of massive spanning strata in longwalls or pillar extraction (Rice 1935). In addition, there have been many cases of rockbursts and coalbursts occurring in response to the seismic events created by slip along pre-existing geological structures. This has occurred at distances of up to 100-120 m from the structure in coal mines in the USA (Heasley et al., 2001). Seismically triggered bursts are suggested to cause significant dynamic shear stresses in the coal resulting in dynamic failure of the coal (Iannacchione and Tadolini 2015). However, it is suggested that the coal failure mechanism with seismic shock is actually more complex.

Dou et al., (2009) explained the cause of seismic events triggering coalbursts as a function of strong-soft-strong strata. When there is stiff, strong massive strata, it is capable of transmitting seismic energy very efficiently, with minimal energy loss or absorption. When a seismic wave passes from a strong stiff unit into soft strata, the soft strata does not transmit the seismic energy as efficiently and acts as an energy absorber.

![Figure 4: A simplified explanation of the seismic shock mechanism (Wang et al., 2015)](image)

The loss of confinement mechanism, identified by Babcock and Bickel (1984), occurs as a result of the rapid loss of coal pillar confinement between the coal and roof or coal and floor or by reducing the confinement on highly stressed coal by mining into a pillar core in burst prone strata. This mechanism was demonstrated in their own laboratory tests on coal samples. Their findings are consistent with similar results presented by Brauner (1994) which demonstrated that bursting would occur around a hole (Figure 5) drilled into a sufficiently triaxially loaded sample of coal (between 156-41 MPa). For this mechanism to occur high stresses need to be present on the excavation boundary and this typically
requires high shear strength between the coal and roof or floor, which is known to occur in burst prone mines with rough irregular contacts between the coal and sandstone in the USA. “Red coal” (mylonite) is often observed on the failure planes which are left, following a coalburst.

Figure 5: Borehole coalburst, an example of the loss of confinement mechanism (Brauner 1994)

CONCLUSIONS

One thing which is clear from the international experience of coalburst and mining seismicity, is that once it starts to occur it is not something which will go away. It is a risk which causes injuries and fatalities, and the survival of individual mines which experience seismicity is purely a function of the rapid response to the risk through adequately resourcing and establishing technical expertise in order to implement high quality management and engineering controls. At this point in time, in Australia, where we have very little experience and technical expertise in coalburst management, this is likely to prove a formidable challenge. However, the Australian coal industry has demonstrated an exceedingly low tolerance for fatal risk and an internationally enviable safety record (Harris et al., 2014). Substantial research and development funding is already being committed to address the risk through the Australian Coal Industry Research Program (ACARP). The success in managing outburst is clear evidence of the capability of the Australian coal industry to address the risk of coalburst without having similar multiple fatality mining disasters which have occurred overseas.

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REFERENCES


COALBURST CONTROL METHODS

Justine Calleja\textsuperscript{1} and Ian Porter\textsuperscript{2}

\textbf{ABSTRACT:} Coalburst (also known as coal bump) is a well known phenomenon in underground coal mines internationally, however, it was not recognised as a risk for Australian coal mines until the recent double fatality at Austar Coal Mine in the Hunter Valley in 2014. This paper reviews the international knowledge base from research and practice to provide Australian mining professionals with an understanding of the basics of coalburst control methods in order to allow mine operators to address the risk of coalburst in mining safety management plans. This is the second of two companion papers to be read with the first paper, “Coalburst causes and mechanisms” (Calleja and Nemcik 2016).

\textbf{INTRODUCTION}

There are many different methods available to manage coalburst risk. The approaches can be broadly categorised into predictive, preventative, mitigating and protective control measures. Control measures range from very simple rules of thumb to highly sophisticated technology and engineering. The most effective coalburst management systems are likely to include a mix of both approaches.

\textbf{PREDICTIVE CONTROL METHODS}

Predictive control methods can be used to identify coalburst risk prior to mining in order to allow mine planning and design changes to be implemented in order to reduce or eliminate the risk (planning prediction or strategic controls). Predictive measures can also be used during mining where coalburst risk has not been eliminated, in order to minimise the hazard through the implementation of mitigating and protective control measures (operational or real time prediction).

\textbf{Planning Prediction}

The first and most important predictive control measure, which will determine the need for other control measures is risk assessment. Site specific empirical risk assessment has been used successfully at the Lynch No. 37 Mine in Kentucky where the key risk indicators were found to be depth greater than 460 m, sandstone thickness greater than 10 m and sandstone in contact with the top of the coalbed (Iannacchione and Tadolini 2015). However, these risk factors and values may not be appropriate for use at other mines as they are a function of the specific geotechnical and mining environment present at this particular site.

Mark and Gauna (2015) recognised that a universal quantitative risk rating scale has not been developed, but suggested rating the individual factors which are known to be important, based on existing cases, on a site by site basis with regard to existing experience at the site. The risk analysis factors suggested for pillar recovery which would be considered high risk include depth of cover greater than 450 m, multiple seam workings with remnants surrounded by goaf, strong thick and massive strata (e.g. a Coal Mine Roof Rating Unit Rating > 50 (Calleja 2006, 2008) within 15 m of the seam, presence of geological features (such as sandstone channels, faults, fracture zones, seam dips or rapid topographic changes), panel width larger than 150 m, full extraction and a burst history at the mine.

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As there are currently no risk assessment guidelines applicable for use in Australia, empirical risk assessment can be conducted based on the risk factors described by Calleja and Nemcik (2016). The most important risk indicator is a history of coalbursts in the seam and at the mine, as many of the other factors associated with coalburst can be present at mine sites which do not ever experience coalbursts (e.g. seismicity, high depth of cover, presence of thick massive strata in proximity to the seam). Mark and Gauna (2015) provided evidence from eleven mines that a past history of bursts is an important indicator of burst risk, “Major bursts have often been preceded by a pattern of increasing coalburst activity.” The development of Australian coalburst risk assessment guidelines is currently of critical importance. Until such guidelines are developed, it is recommended that the principal indicator to use to determine the presence of coalburst risk should be a past history of coalbursts, and the Mine Safety and Health Administration (MSHA) (2012) guidelines for reporting coalbursts are a reasonable criteria to use.

In the Czech Republic, risk assessment is based on both regional risk and local risk factors (Ptacek, 2015). Regional risk factors include tectonic features, seismic and microseismic monitoring, stratigraphic sequence, mechanical properties of the coal and rock, depth, seam thickness and dip. Local risk factors are used to determine specific areas within a mine which may be at risk and include depth, seam thickness and mine layout (mining next to goaf or under remnant pillars).

Dou and Gong (2014) described the zoning and levelling forecasting method of predictive control (see Figure 1) which is used in Chinese coal mines. It is an integrated approach which involves early regional risk assessment combined with real time local and point forecasting of coalburst. This predictive system uses the comprehensive index method, the multi-factor coupling method, microseismic monitoring, the electromagnetic radiation method and the drilling cuttings method.

![Figure 1: Zoning Spatio-Temporal forecast method of prediction (Dou and Gong 2014)](image)

A great deal of research effort has been focused on developing methods to predict the location and timing of coalburst. Traditional methods of prediction have involved the use of test drilling and incident history recording. More technologically advanced systems under current investigation include the use of seismic monitoring, seismic tomography, electromagnetic radiation (EMR), stress and strain monitoring and numerical modelling.

Tan et al., (2014) described a method of coalburst risk monitoring which was used at Yangcheng Mine in China. Three different monitoring strategies were employed concurrently and indicated that a silence period in the microseismic monitoring, combined with increasing electromagnetic radiation could be used to predict a coalburst. Once the risk was identified, face drilling with fines monitoring could be conducted to identify the location of high burst risk in the face and allow targeted de-stressing measures to be implemented.
Traditional Methods – Observation, Rock Noise Reporting and Test Drilling

Traditional techniques are often inherently simple and inexpensive. The value of these approaches, such as rock noise reporting and event histories, should not be overlooked as useful methods of assessing coalburst risk.

The use of qualitative (observational) seismic monitoring is still an important component of seismic management systems. Mt Isa Mines initially relied on “Rock Noise Reporting” by underground operators to monitor seismicity in their Deep Lead orebody (Thin et al., 2005). Similarly, seismic rock fall reporting and rock noise reporting (a tick and flick form) were introduced for seismic management when Beaconsfield first encountered mining seismicity in 2003 (Hills 2013). In both cases the level of seismicity increased and required an instrumented seismic monitoring system to be installed. In the Czech Republic, Ptacek (2015), recommends that the data necessary for continuous coalburst prediction includes personal monitoring, by miners, of rock strata and coal seam behaviour.

High static stress conditions have been proven to be capable of causing coalbursts even without external seismic events (Brauner 1994). As such, being able to identify high in-situ stress areas and the development of high stress in the coal close to the excavation boundary is a key tool for identifying areas of coalburst risk. Regular stress mapping by trained geotechnical personnel can be conducted to identify and predict high stress areas. It is very difficult and expensive to measure triaxial in-situ stress and stress change in coal with existing instruments. Whilst such stress measurements are very important and valuable for geotechnical design they do not provide the level of areal coverage required to identify coalburst risk zones. Test drilling is a traditional, proven, simpler and cheaper method of identifying high stress areas with better areal coverage in coal, in advance of mining, and it is used widely in China and the Czech Republic (Ptacek 2015; Konicek 2013 and Li et al., 2007). When a borehole is drilled in highly stressed coal, it will burst during drilling and a significantly larger volume of coal cuttings will be washed out than would occur when the coal is not highly stressed. This process is often accompanied by the observation of a bumping noise in the coal while drilling and shocks on the drill rods. Brauner (1994) provided an example of drill cuttings from a 46 mm hole which produced 2-4 l/m cuttings under normal conditions and which increased to 15-22 l/m at the beginning of a high stress zone beyond 4 m into the rib line where dynamic effects occurred. It is also possible to identify high stress zones from the axial forces required to be applied during drilling. High stress areas prone to bursting can be identified when the drill rod is pulled into the hole and axial forces on the rods go into tension (Figure 2).

\[ \text{Figure 2: Yield of cuttings in boreholes approaching a high-stress zone and the axial forces on the drill in coal under normal (above) and high stresses (below) (Brauner 1994)} \]
This approach could be applied to the longhole drilling which is conducted for gas drainage in order to identify coalburst risk areas after gas drainage has occurred, however, the application of this approach will need to be developed based on underground trials in Australia.

The level of static stress can change as a result of mining and new areas of high risk can develop after initial test drilling has been conducted. As a result repeated test drilling is sometimes required.

**Seismic Monitoring**

Ortlepp (2005) recognised that whilst seismic monitoring and numerical modelling could not provide real-time prediction of rockbursts, it would be likely to allow prognosis (risk identification). However, this would require adequate staffing of a sophisticated monitoring and analysis system by a dedicated experienced professional for mining operations experiencing bursting. Mendecki et al., (1999), describe the applications of seismic monitoring. It can be used to identify the location of possible coalbursts and can help guide rescue operations. It can be used to identify the location of important geological features such as faults and dykes ahead of mining. It can be used to identify unexpected changes in the location and timing of seismicity in the mine which can indicate increased risk and allow management to implement safety management and strategic controls. It can also be used to allow characteristic patterns of seismicity which precede coalbursts to be identified and used to help predict the potential triggers, location and timing of future incidents. It is a useful input to review the effectiveness of planned mine designs and production methods in managing seismicity.

Li et al., (2007) highlighted the importance of seismic monitoring systems for managing mining seismicity in underground coal mines in China: “We find that the key problems of rockburst (coalburst) hazard mitigation in China are the lack of mine seismicity-monitoring networks in most mines, and the need for improvement of the accuracy of monitoring systems for mines that have been equipped with such systems.”

A seismic monitoring system consists of transducers, data acquisition hardware and processing software. Geophones are useful sensors for larger spacing (e.g. 1 km) and surface installation whereas piezoelectric uniaxial or triaxial accelerometers are ideal for more closely spaced networks (e.g. 100 m). A single short period earthquake monitoring instrument should be installed in a mine network to allow data recovery of the largest events, which are not well recorded by geophones or piezoelectric sensors. A Global Positioning System (GPS) is employed to time-link regional and local systems (Urbancic and Trifu 2000).

The type of monitoring system required to manage mining seismicity is different from the approach applied both for monitoring earthquake seismicity and microseismic rock fracturing research investigations. In Australian and Canadian hard rock mines the inter-sensor spacing of seismic monitoring systems is mostly less than 200 m and the layout of these systems is substantially different to the systems used in South African gold mines which have sensors at 300 - 650 m spacing (Potvin and Wesseloo 2013). In order to manage mining seismicity the seismic monitoring data needs to be collected and analysed continuously in real time. The microseismic monitoring systems which have been used regularly in the coal industry to understand rock fracturing associated with mining would need to be modified to be used for seismic risk management.

**Electromagnetic Radiation**

It has been found that the deformation and failure process of coal generates both acoustic emission and EMR. Dou et al., (2001 and 2004) demonstrated that the EMR intensity increased sharply in the case of burst failure of coal. Wang et al., (2012) found that the EMR signal was linearly related to the applied loads. It was thus inferred that monitoring the EMR signal could be used to reflect the stress state of the coal rock mass. EMR intensity was monitored in a longwall from 10 locations in the tailgate and ten locations in the maingate in 2012. The monitored mean of maximum daily EMR intensity measured at
two points in the tailgate located 200 m and 220 m from the longwall face was more than double the background levels 10 days before a coalburst occurred, and peaked at 5 times the background level one day prior to a burst which occurred at a distance of 178 m from the longwall face (Figure 3). This data was consistent with previous research results which indicated that EMR monitoring can be used to assist in predicting bursting. (Dou and Gong 2014)

![Variation of measured EMR intensity in the tailgate of a longwall face before and after a coalburst on June 26th, 2012](image)

**Figure 3: Variation of measured EMR intensity in the tailgate of a longwall face before and after a coalburst on June 26th, 2012 (Dou and Gong 2014)**

**Seismic Velocity Tomography**

The seismic p-wave velocity in rock is known to vary as a function of the *in-situ* stress conditions (amongst other factors, such as density and porosity). Geophones can be used to monitor the arrival times of seismic waves and then calculate the velocity of seismic waves through the coal between the seismic source and the geophones. When multiple geophones are used, areas in the seam with higher and lower seismic wave velocities can be determined by using Computed Tomography (CT) and seismic CT plans can be generated. Areas of higher coalburst risk were shown to occur in high velocity or high velocity gradient areas, by Lurka (2008), in Zabrze Bielszowice Coal Mine in Poland. This method has since been applied in more than ten coal mines in China with useful results (Dou and Gong 2014). Wang *et al.*, (2015) used Seismic Velocity Tomography (SVT) at Xingan Coal Mine in China and found that the development of high velocity regions in the coal, when located close to the coal face was a predictor for coalbursts.

**PREVENTATIVE AND MITIGATING CONTROL METHODS**

The most effective method of addressing risk is through elimination. In many cases where severe levels of coalburst risk have been recognised, the risk has been eliminated by avoiding mining in the high risk area, or by modifying the mine plan or extraction method. The Lynch No. 37 Mine in Kentucky chose to avoid longwalling through areas they had identified as high risk (Iannacchione and Tadolini, 2015). In other cases, bursting risk has been reduced through mine design by avoiding mining underneath remnant pillars, by using barrier pillars, by using yield - abutment gateroad pillar combinations, by avoiding the creation of goaf corners or by reducing the mined panel width. Numerical modelling and stress analysis are valuable tools that assist in identifying mine designs that will create high risk stress concentrations which may cause coalburst and assess the potential improvements available through alternative design approaches. Other measures to reduce coalburst risk, such as de-stress drilling, blasting and water infusion have been used in Europe and China.
Provocative Blasting (Pre-Conditioning)

Provocative blasting (pre-conditioning) is a technique which has been used primarily in hard rock mines, where blasting is the normal excavation method, to reduce the risk of development face bursts and was very successful in hard rock at Mponeng, South Africa (McGill 2005). It involves drilling and loading a number of holes which are longer than the production holes, and blasting them prior to the production holes so that the new face created will be fractured and unable to store the necessary elastic strain energy required for bursting. De-stress blasting in coal is one of the oldest methods of coalburst prevention. Blast holes are drilled around the development face or into the longwall face and are designed to fracture the coal and seam contacts in place rather than throw it. However it is not as effective as de-stress drilling and it is not amenable to routine application in high productivity longwall mines as holes need to be blasted every 2.5 m advance (Brauner 1994). It is currently used in The Czech Republic. Konicek et al., (2013) describe a case where 42 mm holes 11-15 m long and at 5 m spacing were drilled and charged with 7-9 kg of explosives at the Lazy Colliery.

Long Term Water infusion

Long term water infusion is used to increase the moisture content of the coal which reduces its liability to bursting. During borehole - bursting laboratory tests it was found that fully saturated coal samples would not burst (Figure 4). In practice a 1-2% increase in moisture content may be sufficient to eliminate burst danger. The limitation on this method is that infusion may need to be conducted for between a few days and several months with fluid pressure between 1-40 MPa, and it may not work at all in existing highly stressed areas where it is difficult to drill and where the coal permeability is very low. However, the method is significantly more amenable to high productivity longwalling than roadway de-stress blasting and de-stress drilling and it has been used successfully in the past to prevent coalbursts (Brauner 1994). It is currently used as a standard technique to prevent coalbursts in the Czech Republic (Konicek et al., 2013).

![Figure 4: Different modes of stress decrease in coal specimens during drilling](image_url)

In Figure 4 the Curve I sample was air dried coking coal and experienced six borehole bursts. Curve II was the same coal, but saturated with 2.4% moisture content. The Curve III sample was anthracite. No bursting occurred for the Curve II and Curve III samples (Brauner 1994).
Hydrofracking

Hydrofracking involves the injection of water into strata at high pressure to induce fracturing. Hydrofracking can be used to create fractures in the coal, to prevent it from being able to store sufficient elastic strain energy to burst. For example, 10 m long holes at 5 m spacing are drilled into the face and injected with water at 40 MPa for 20 minutes. A decrease in water pressure of 10 MPa or more can be indicative of adequate de-stressing. This method is suggested by Brauner (1994) to be the most economical de-stressing technique. Hydrofracking can also be used to fracture the strong stiff strata which is required to develop high stress concentrations and transmit seismic energy to the coal to trigger coalburst.

De-Stress Blasting

De-stress blasting of stiff, massive overburden strata over the longwall block is an important coalburst control technique which has been used successfully in the Czech Republic since the 1990s and internationally. Konicek et al., (2013) described its application at Lazy Colliery where 43 mm boreholes were drilled at a 10 m spacing along the gateroads into the longwall overburden strata for a length of 40-100 m.

De-Stress Drilling

De-stress drilling is a very successful method of coalburst prevention which has been used in Germany and Russia. 95 - 600 mm diameter holes are drilled into the area of highly stressed coal at a spacing of between 0.5 m and 10 m and in the order of 6 - 10 m long depending on conditions. The holes will burst and fines are removed during drilling until the stress has reduced below bursting levels. It has to be repeated every 5 m or so of advance, so although it is effective, it is a slow process and is not amenable for high productivity mining.

PROTECTIVE CONTROL METHODS

Protective control measures have been used to prevent coalbursts from causing injuries. Some examples of these include the use of flippers and rubber mats on longwall shields, and the use of blast proof barriers on continuous miners (Mark and Gauna 2015). However, these measures are only useful for protecting miners against small coalbursts, and given the difficulty in predicting when and where a coalburst will occur, let alone whether it could be a large one or a small one, such measures should not be relied on to prevent injuries and fatalities.

If a coalburst is anticipated then a much more reliable approach is to remove people from the area at risk through the use of remote mining methods such as remote control cutting or drill and blast mining (grunching). The practice of specifying re-entry times is a common and successful method to separate miners from the area of risk after production blasts, cuts or de-stressing measures which can be developed based on seismic monitoring data (Hills 2013).

Dynamically rated (yielding) support systems can be used to minimise excavation and machinery damage, but the engineering design approach is not sufficiently advanced at this time for support systems to be adequate to protect people from coalbursts (Heal 2010). Dynamic support systems are designed to absorb high levels of kinetic energy and maintain support load whilst allowing large deformation (e.g. 200 mm). Yielding integrated support systems are composed of surface confinement (mesh, steel fibre reinforced shotcrete, laced cables), high strength anchoring bolts (fully encapsulated rebar and cables) and displacement capable bolts such as split sets, cone bolts and de-bonded cables (Heal 2010 and Hebblewhite et al., 1998). Heal (2010) dynamically tested thirteen yielding support systems and found the support system is only as strong as its weakest link which is usually the surface support (e.g. mesh), or connections between the surface support and bolts or cables.
CONCLUSIONS

Extensive research work and application of coalburst control methods has been undertaken internationally, however the applicability of these approaches will need to be tested in Australian conditions. Additional work will also be required to modify existing techniques to be amenable to the high productivity longwall mining which is conducted in Australia. The existing coalburst control methods can be broadly categorised into predictive, preventative, mitigating and protective measures. Ultimately successful coalburst management will require the ongoing development and refinement of operational seismic management plans which take a holistic approach to risk identification, risk monitoring and the application of risk management control measures.

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REFERENCES


A PRACTICAL APPROACH TO THE DETERMINATION OF OUTBURST THRESHOLDS

Ian Gray, Iuliia Shelukhina and Jeff Wood

ABSTRACT: The current Australian reliance on gas content and DRI index used in Outburst Management Plans leads, in many cases, to an ultra-conservative approach to coal mine development. It is also contrary to overseas practice where other factors are taken into account. Outbursts must start with failure and that failure must produce fragments. It is therefore a process of fragmenting failure. For this to occur the coal must have some pre-existing structure within it. Determining whether a fragmenting failure will take place is dependent on the effective stress within the coal and the strength of the coal. Thus determining the strength of the coal and the structure within it are paramount, both on the small and large scales. A key element in the effective stress equation is the gas pressure. Once failure has occurred the question is whether that failure takes place with sufficient energy to provide a serious risk? The energy comes from gravitational effects, strain energy and expanding gas. The energy from expanding gas tends to dominate and is dependent on gas in pore space and desorbing gas. The desorption rate of the latter is dependent on the diffusive characteristics of the coal, the gas content and the fragment size. To be able to practically characterise a coal for its outburst proneness it is necessary to measure more than is current practice in Australia. What is required is determination of structure at various levels, the measurement of diffusion coefficient like behaviour, strength by Protodyakanov Index, or by rapid depressurisation (Pop Gun Test) as well as gas content.

INTRODUCTION

The purpose of this paper is to present an improved approach to the determination of outburst risk compared to current Australian practice. The need is to ensure safety in mining but not to prevent mining in circumstances that would present a minimal threat.

To be able to do this the paper endeavours to explain what an outburst is. It also examines current Australian practice and compares it to that used overseas. It then looks in a more fundamental level at the outburst process and the energy release that occurs during these events. Using a combination of practical tests, observations and theory it is possible to arrive at alternative approaches to those in current use in Australia. These provide a sounder basis than those currently used. These approaches are not however based on a simple single parameter test.

This paper is to a significant extent a synopsis of the work contained in the ACARP project C23014 (Wood and Gray 2015) and reference should be made to that substantial document and its appendices to gain the full description of the work and conclusions.

WHAT IS AN OUTBURST?

An outburst is a violent expulsion of coal and gas from a working face. Sometimes rock is also dislodged in the outburst. An outburst is a process that follows failure of the coal or rock. Whether the failure then transforms into an outburst is very much dependent on the mode of failure, the gas storage and gas generation during that process. Outbursts cause fatalities by two mechanisms; the first by mechanical injury caused by moving particles and gas, whilst the second by asphyxiation due to displacement of air by the gas evolved. Outbursts also occur from rock, notably porous sandstones and porous or vuggy salt deposits. It is suggested that the division between outbursts, rockbursts and gravitational slumps can be determined by plotting the potential energy release on a ternary diagram as shown in Figure 1.
In examining what is a very extensive amount of literature on outbursts in coal from around the world it is apparent that the worst outbursts in coal have occurred during the entry from rock into sheared coal associated with faulting. Indeed Russian standards specifically require greater care to be taken on entering a seam as compared to driving a roadway in seam.

![Diagram of outburst types](image)

**Figure 1: The proposed basis of determining whether an event is an outburst, rockburst, or slump based upon the potential energy components**

One large outburst occurred at Luling mine in Anhui province, China on 7 April 2002. It involved rock drivage into a coal seam at the location of a fault. It produced 8,730 tonne of coal and rock and 0.93 million cubic metres of gas. A sketch of the mining situation at Luling is given in Figure 2 and the result in terms of coal and rock ejected is shown in Figure 3. A similar case was that of Sanhui Mine in Sichuan where on 8 August 1975 an outburst occurred that produced 12,800 tonne of coal and rock with 1.4 million m$^3$ of gas. In both cases the gas was methane.

Large outbursts do also occur with mining in coal. On 25 April 2007 one occurred at Dashucun Mine in Hebei province. It involved mining in No 4 seam which should have been destressed and degassed by the mining of an adjacent seam. This did not take place because complex faulting had meant that a pillar was left in place. When No 4 seam was mined in the stressed environment 1,270 tonne of coal and 9.3 x 10$^4$ m$^3$ of gas was released in an outburst.

The Sanhui mine outburst followed shotfiring while the Luling case followed the use of an air pick. Outbursts of this size can readily lead to the total loss of a mine, as there is gas to ignite and coal dust to explode.

Many of outbursts that are severe occur on geological structures in the coal seam that contain gouge (ground up) material. Some however occur from solid coal which fragments during the outburst. Examples of two kinds of outburst can be seen below.

Figure 4 shows a sketch of an outburst that occurred at Westcliff Colliery, NSW which moved the continuous miner backwards. The energy source of this outburst was a sheared zone of coal behind the face.
Figure 5 shows a sketch of a typical outburst that occurred from solid coal at Leichhardt Colliery, Queensland. Here the outbursts always occurred across the cleat, often preceded by an onion ring appearance in the face before buckling occurred outwards leaving a cone in the ribside. The size of these outbursts varied from 1 to 350 tonnes.

For an outburst to occur, failure of the coal must first take place. Failure is commonplace in mining and is due to the effective stress exceeding the material (in this case coal) strength. In an outburst the failure is accompanied with the release of energy and gas. The key to understanding outbursts is determining the likely sources of energy release while the key to controlling them is in minimising the potential for energy release.

In their paper Black et al., (2009) show some relaxation to the gas contents (Figure 6) for Tahmoor and Westcliff Collieries. In the case of Tahmoor some note is taken of coal structure.

To complicate matters further GeoGAS introduced the Desorption Rate Index (DRI) as an indication of outburst proneness. This test involves the taking of a core for gas content measurement as per quick crush measurement (AS3980-1999). There is some initial but variable gas loss before the core is placed...
in the canister. This is followed by additional gas loss while the initial rate of gas desorption is determined. The canister is then sealed and the gas content of the core approaches some state of equilibrium with the partial pressure of the gas in the canister. This is dependent on the amount of coal in the canister, its gas content prior to being placed in the canister, the dead volume of the canister and in addition the temperature of the canister. At the laboratory the canister is drained of gas, the volume of which is measured, and the core removed.

Figure 4: A plan of an outburst that occurred at Westcliff Colliery (Adapted from Marshall et al., 1980)

Figure 5: A typical view of the ribside after mining through small outburst cone at Leichhardt Colliery (Adapted from Moore and Hanes 1980)

A sample of approximately 200 g is then taken from the core and crushed (in a specific crusher) for 30 seconds and the gas volume that is released is measured. The gas release during this process will depend on the gas content of the coal taken from the canister, the degree to which the core breaks up on crushing and the diffusional behaviour of the coal fragments.

If the sample is of a different mass than 200 gm the volume released is corrected by ratio to this mass. The DRI value is then calculated by multiplying the volume released during the 30 second crush by the
ratio of the total gas content from the full desorption process to the gas released from the 200 g sample during the 30 second crush.

This final correction tends to force all the DRI values to follow the gas content. Hence the linearity of the plot shown in Figure 7 which is used to justify the use of the DRI process. In Figure 7 it is possible to see that the value of DRI of 900 ml in the first 30 seconds of crushing corresponds to an initial gas content of 9 m$^3$/t of methane or 6 m$^3$/t of carbon dioxide in Bulli seam coal. These gas content values are Lama's (Lama, 1995) estimates of the gas contents that lead to outbursts. This is the justification by GeoGAS for the use of the value of DRI 900 as an indicator of outburst conditions.

The process of arriving at a DRI value is described further below with reference to the isotherms shown in Figure 8. The steps shown are:
1. The initial gas content and pressure
2. The gas content following some loss on coring
3. The gas content following further loss due to Q1 sampling
4. The drop in gas content and pressure as an equilibrium is approached between coal gas content and the canister pressure.
5. The gas content at 1 atmosphere partial pressure of gas

The 30 second quick crush of the DRI approach obtains some of the gas between points 4 and 5 and then multiplies it by the ratio of the total gas content/gas released in 30 seconds.

![Figure 8: Methane and carbon dioxide isotherms showing the stages of pressure/volume drop through the process of gas content determination](image)

Worryingly there has been a trend to use the DRI 900 value to determine whether non Bulli seam coals are outburst prone. This compounds the problems of measurement error with inconsistencies between coal seams. It is the opinion of the authors that the DRI 900 measurement is an unreliable indicator of outbursting conditions that is founded on pseudo-science fitting a straight line to some group of data without adequate thought to the measurement process and the errors it contains.

**OVERSEAS OUTBURST RISK THRESHOLD DETERMINATION PRACTICE**

The basic practice of outburst threshold determination may be broken into several categories. These are:

1. A determination of mining area proneness obtained by experience
2. Examination of the geology with particular reference to structure
3. Tests conducted locally to determine parameters which can be broken into:
   a. Pressure measurement
   b. Toughness measurement
   c. Drilling tests
   d. Gas desorption indices

The process of determining the outburst proneness by experience is one that most miners would prefer to avoid.

The careful examination of geology is important. Changes in coal structure are often the key to conditions becoming outburst prone. The Chinese description of coal structure is useful here. It is shown in Table 1.
Gas pressure measurement is regarded as being of particular importance by the Chinese. Pressure is a fundamental parameter and great care is taken to measure it. This is done by drilling cross measure and cementing a pipe in through stone which is then connected to a pressure gauge. While some outbursts have been reported from 0.61 MPa gauge the threshold value for outbursting is considered to be 0.74 MPa gauge. The gas pressure threshold used in the coal seams of the Pechorskiy basin, Primorie and Sakhalin Island in Russia, is 10 kgf/cm² (1 MPa).

Table 1: Chinese strength definitions of structurally affected coal

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Structural Mode Name</th>
<th>Structural Features</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Unbroken Coal</td>
<td>Layered and blocky structure;</td>
<td>Hard;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Strips are obvious.</td>
<td>Hard to break by hand.</td>
</tr>
<tr>
<td>II</td>
<td>Broken Coal</td>
<td>Layered structure;</td>
<td>Medium hard;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Strips are obvious with movements;</td>
<td>Easy to break by hand.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Irregular shape with angle;</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compression properties.</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>Seriously broken coal</td>
<td>Tectonic lens structure;</td>
<td>Low hardness;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Small and schistose structure;</td>
<td>Easy to break into powder by hand.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fragments.</td>
</tr>
<tr>
<td>IV</td>
<td>Comminuted coal</td>
<td>Cemented small particles.</td>
<td>Occasionally hard hardness;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Easy to break into powder by hand.</td>
</tr>
<tr>
<td>V</td>
<td>Pulverized coal</td>
<td>Soil structure;</td>
<td>Loose;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gouge materials</td>
<td>Easy to break into powder by hand.</td>
</tr>
</tbody>
</table>

Toughness measurement is somewhat different in that it is not a measurement of a fundamental parameter of toughness which can be defined as the energy required to cause rupture per unit volume. This is difficult to achieve. What is used instead are two tests that are index tests. Index tests are tests that make a measurement that is dependent on several fundamental parameters. In the case of coal toughness, two tests are used, both of Russian origins. The first is the Protodyakanov Index which is used both in Russia and extensively in China and involves the use of a simple drop hammer. The second is a penetrometer gun that is used to test the toughness of coal plies underground.

The Protodyakanov index is measured by drop hammer test. This is a test on lump coal with measurement of the coal size reduction. The process involves four weighed sets of coal consisting of 5 subsamples each with size range 20 to 30 mm and weight 40-60 g. The subsample is placed in an apparatus (Figure 9) comprising a drop hammer of 2.4 kg weight with a 600 mm travel. The diameter of the hammer is 66 mm and the tube it falls within is 76 mm. The number of hammer blows depends on coal strength and is determined experimentally. The amount of fines of less than 0.5 mm diameter is measured in a measuring cylinder (a tube of 23 mm diameter). The height of fines in the measuring tube after crushing of one set (5 subsamples) should be in the range of 20 – 100 mm, otherwise the number of blows should be adjusted experimentally. For the coal usually one blow is enough, but for some strong coals 2-3 blows are required.

The $f$ coefficient is defined by equation 1:

$$f_{20-30} = \frac{20 \times n}{h}$$ (1)

where $f_{20-30}$ is the toughness index (for 20 to 30 mm size range), $n$ is the number of hammer blows, $h$ is the scale measurement in the cylinder after 5 subsample tests (mm).

The final result is an average of 4 measurements.

Slastunov reports that there is quite a wide variation in the $f$ value of outburst and non outburst prone coals but that a value of $f$ less than 0.54 indicates a high likelihood of outbursting in the Kuzbass. The general threshold value used in China is 0.5.

The Chinese extension to the test method for fine coal where it is not possible to obtain 20 to 30 mm lumps is to sieve the sample for the 1 to 3 mm range. This is then hammered three times and the size reduction noted by a measurement in the fines cylinder.

In this case, if $f_{1-3} > 0.25$ using equation 1 with $n = 3$ then the equivalent

$$f_{20-30} = 1.57 \times f_{1-3} - 0.14$$ (2)

Otherwise if $f_{1-3} \leq 0.25$ then $f_{20-30} \equiv f_{1-3}$ (3)

**Russian penetrometer gun – Index q**

This is a spring loaded penetrometer system that has to be wound up and fired into coal at the face. A number of tests need to be undertaken in each ply. Tests within plies need to be averaged and different plies show quite different results. The probe is designed to penetrate with energy of 27 J, however the mass and velocity are unknown. The device is shown in Figure 10. The $q$ index is calculated from the penetration of the probe punching into the seam according to equation 4. The penetration result is determined from the average of 5 measurements with 5-10 cm distance between locations.

$$q = 100 - l$$ (4)
where, \( q \) is the value of the \( q \) index, \( l \) is the depth of the probe penetration (mm).

If \( q \leq 75 \) the ply is considered to be outburst prone.

Figure 9: Drop hammer equipment – measuring cylinder with scale and tube with drop hammer

Figure 8: Russian penetrometer gun for \( q \) index test

The Russian literature seems to consider that there is a correlation between the \( f \) index derived from the Protodyakanov Hammer and the \( q \) value from the penetrometer gun. This relationship is given in equation 5, though the correlation is not exact.

\[
f = 0.4 \quad q (110 - q)
\]

(5)

Cuttings volume determination

Chinese, Russian and Kazakh operators use a measure of the cuttings volume generated on drilling as an indication of outburst risk. The Germans also used this system when they mined underground. The system did not however work when tried at Leichhardt Colliery in Queensland. The general idea is that a hole of a known diameter is drilled using air flush or a scroll (auger) drill and the volume of the cuttings
generated is measured and compared with the theoretical hole volume cut. The volumetric difference may be very great.

Cai and Luan (1995) report the drilling of a 43 mm hole. The nominal volume of such a hole is 1.45 litres/m. The partial outburst threshold is considered to be 7 litres/m of coal corresponding to an over break to a nominal diameter of 94.5 mm and is five times the expected hole volume. Slastunov (2014) reports French and Belgium experience where the over break volume in outburst prone areas was 5 to 8 fold for 115 mm diameter holes while in non-outburst prone zones it was 2 to 3 fold. The latter seems to be a large value.

**Borehole Flow Tests**

Another test used in China, Russia and Kazakhstan are those where a section of hole is drilled and a packer system inserted. The flow per unit length is then measured shortly after drilling. This test provides the desorption value, g (l/min). These tests have been used in extremely impermeable coals and seem to be more of a measure of the diffusion rate from the hole wall than a measurement towards deriving permeability. Indeed high permeability coals are less outburst prone because they drain ahead of the face provided that the advance rate is not too fast.

**Desorption Indices**

A whole series of desorption tests have been developed for use on cuttings derived from drilling. Most of these are of the form where cuttings are collected within a certain period from drilling using a hand held auger (scroll) drill. They are then sieved to a size range and this is sealed within a container for a certain period after drilling. The desorption rate or pressure rise corresponding to it is then measured over this short period and regarded as being an outburst index. This is the nature of the Chinese Drilling Cuttings Desorption index (CDCDI) parameters. This yields the Parameters $\Delta h2$ and $K1$. The Polish mines use a virtually identical instrument (Lunarzewski 1995 and Lama 1995 b). The Chinese have also developed an electronic instrument for a similar measurement. Indeed the concept of these tests is not far different from the Hargraves’ emission meter used in Australia until the early 1980’s.

The main problem with these desorption tests is that they look at a snapshot of the desorption process on a very controlled size range. The measurement period is not enough to define gas content or diffusional behaviour.

Another type of testing that was in vogue consisted of several kinds of adsorption tests. In these the samples uptake of gas was measured. One form of uptake test, the $\Delta P$ index, is used in China and forms the basis of one of their outburst proneness determinations.

**Combinations of tests**

Cai and Luan (1995) proposed the combined approach of these three measurements as being indicative of outburst conditions. Their criteria were that the volume of cuttings from a drilled length of a 43 mm diameter hole was 7 times the nominal drilled volume. In addition the initial measurement of gas flow rate per unit length of hole per metre within 2 minutes of drilling exceeded 3.8 litres/minute. Finally that $\Delta hz$ gas desorption volume from 1 - 3 mm coal cuttings exceeded a set value.

Zhang (1995) describes another combined approach which is part of current Chinese outburst determination practice. In this Outburst conditions are considered to have been reached if $K$ reaches some value.

$$K = \frac{\Delta P}{f}$$  \hspace{1cm} (6)
Where, $\Delta P$ is the initial speed of desorption from coal and $f$ is the Protodyakonov Index.

It is not the normal $\Delta P$ which is measured on absorption. This is presumably a function of gas content, particle size and diffusion coefficient and is possibly a pressure rise measurement in a borehole.

Zhang (1995) also describes the D Index which is part of the current Chinese outburst risk determination practice.

$$D = \left( \frac{0.0075H}{f} - 3 \right) (P - 0.6) \quad (7)$$

where, $H$ is the depth (m), $P$ is the gas pressure (MPa), $f$ is the Protodyakonov index on the softest ply of coal.

If $D$ exceeds 0.25 the coal is considered to be outburst prone. The equation is however only consistent over a limited range of variables. Table 2 gives the current legal basis for mining to avoid outbursts in China.

### Table 2: Combined Chinese parameters for discontinuing mining

<table>
<thead>
<tr>
<th>Coal</th>
<th>Structural mode</th>
<th>Initial rate of gas adsorption, $\Delta P$ (mm Hg)</th>
<th>Coal hardness coefficient, $f$</th>
<th>Gas pressure, $P$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thresholds</td>
<td>III, IV, V</td>
<td>$\geq 10$</td>
<td>$\leq 0.5$</td>
<td>$\geq 0.74$</td>
</tr>
</tbody>
</table>

It would appear that where gas pressure cannot be satisfactorily measured a gas content of 8 m$^3$/t has been fairly recently accepted. Measuring gas content in highly gassy coals, that are prone to disintegrate on coring, causes great problems in some Chinese mines.

**Kuzbass - Outburst forecast during coal seam entry**

As many of the worst outbursts have occurred on entry to a coal seam from workings in stone it is Russian practice to take great care before proceeding to do this. In the Kuznetskii (Kuzbass) basin it is practice to measure the maximum seam pressure from a hole through rock into the seam and to obtain core samples for testing with the Protodyakanov hammer. They use equation 8 to combine these two measurements.

$$\Pi_a = P_{g,max} - 14 f_{min}^2 \quad (8)$$

where, $f_{min}$ is the minimum value of the Protodyakonov Index, $P_{g,max}$ is the maximum gas pressure in seam at a given depth, kgf/cm$^2$ (0.1 MPa) and $\Pi_a$ is the parameter determined by the equation.

If $\Pi_a \geq 0$ then the seam in the mining area is considered as outburst prone. This equation has a consistent form.

In the mines of the Rostov region outburst forecast in a mining area is based on desorption rate ($g$), iodine index ($\Delta I$), and the strength coefficient of the coal ($f$). The coal seam is not outburst prone if all of the following conditions apply: $g \leq 2$ l/min; $\Delta I \leq 3.5$ mg/g; $f \geq 0.6$.
The desorption rate \( g \) is measured in 43 mm diameter boreholes drilled from rock to the seam from a distance not more than 3 m to the seam. The desorption rate is measured in two boreholes not later than 2 min after drilling.

In summary it can be seen that these combined parameter assessments include pressure, toughness and sometimes depth and a diffusion rate term.

**STRUCTURE IN COAL**

As part of the recent ACARP Project C23014 (Wood and Gray 2015) an examination was conducted into the structure of apparently solid coal. This was driven by a need to understand the failure process in outbursting. An understanding was sought as to why failure occurred without a surface existing on which fluid pressure could act. Examination of some polished surfaces of Australian coals showed multiple pre-existing fracture planes within the coal. The spacing of these fractures was in the order of a few millimetres but varied substantially with the coal and ply. Figure 9 shows such a polished section. It is obvious from this that a large number of fractures exist within this sample. It is considered that these form the basis of the fragments that form on failure. Importantly they form the difference between failure and the fragmenting failure that characterises an outburst.

A test was devised to determine whether fragmentation on sudden desorption did indeed take place on these fractures. This involved cutting a specimen from very close to the core shown in Figure 9. This was cut in half and one part placed in a vessel where it was pressurised for 3 weeks and then suddenly depressurised. The fragment size was measured. The second half was examined for pre-existing structure. It was found that the fragmentation that took place was of a very similar sizing to the inter fracture spacing. The test equipment used for this purpose has been endearingly described as the Pop Gun.

![Figure 9: Polished section of HQ (61 mm) core showing structure. Vertical dots at 2.5 mm spacing, horizontal dots at 5 mm spacing](image-url)
ENERGY APPROACH

In the work by Gray (2006) for ACARP and revised by Wood and Gray (2015) the total energy available for release was considered to be fundamental in determining the severity of an outburst. The sources of energy for an outburst are considered to be:

- **Strain Energy from Rock and Coal** – This is dependent on the state of stress in the coal and its elastic properties. Very often the state of maximum stress is limited by failure at the face. In the case of outbursts that progressively erode from the face into solid coal, the state of stress varies from that at the face, which is limited by the unconfined coal strength, to that in the virgin condition. Strain energy may also be supplied to an outburst by the inward movement of the surrounding strata.

- **The Expansion of Gas from Free Void Space** – This comes from the adiabatic expansion of gas from the free void space (cleats). It is a virtually linear function of void space and gas pressure. If the coal is water saturated then there is no gas in the cleats to expand.

- **The Diffusion of Gas from Coal Particles** – Gas may diffuse from the coal particles to an intermediate pressure within the failing coal mass in an outburst. In reaching this intermediate pressure the gas can do work. This gas may then further expand adiabatically to provide energy. The key to the energy release is the gas content which is linked to the gas pressure through the sorption isotherm, the coal particle size distribution and the diffusion coefficient. These factors determine the rate of gas release.

There is also significant energy absorbed during the failure process which reduces the total outburst energy. It is related to the toughness of the coal. Toughness is by definition a measure of energy absorbed in causing failure. The approach of examining the energy release components is valuable in determining the important energy contributions to an outburst or slump. In a slump the principal energy contributor comes from gravity alone. The process of determining the level of risk from an outburst is one of estimating the energy release per unit volume of the outburst and the likely volume of coal that may be involved. The latter may in some circumstances be defined by the extent of gouge material that may be affected.

The energy absorbed by coal failure per unit volume is difficult to measure but indications of the coal toughness may come from grindability testing, drop hammer tests or by gassing up solid stressed coal and suddenly releasing the pressure to determine the level of fracturing that may occur. More work certainly needs to be done to quantify the energy consumed in breaking up coal.

Potential energy release calculations have been undertaken for the outburst situation that might have existed at Leichhardt Colliery. The properties and estimated energy release values are summarised in Table 3. As a reference, the kinetic energy that 1 m$^3$ of coal would have if it fell 1 m (0.014 MJ/m$^3$) is marked at the bottom of the table. It is similar to the potential energy release from gas stored in pore space. These values are however dwarfed by the potential elastic energy stored in the coal and the surrounding rock and by the amount of energy that might be released from desorbing coal. The latter is very dependent on the particle size that is created and the diffusion coefficient of these particles. Small particle size and high diffusion coefficients lead to very high potential energy release values. As a caveat the use of Fickian diffusional behaviour has been made for mathematical convenience. The real process is probably something incorporating both Darcy flow and Knudsen diffusion.

DISCUSSION

The current Australian outburst threshold determination is simply based on gas content or a contortion of it in the form of the GeoGAS DRI index. The latter should be removed as it provides no improvement to the assessment of outburst risk and is based on faulty logic.
The basis for outburst risk determination needs to take account of the outburst process. This is one of failure with fragmentation and energy release. By definition of an outburst as opposed to a fall, slump or rockburst, gas is an important contributor. It is suggested that a basis for a definition between outbursts, rockbursts and slumps should be based upon the ternary diagram shown in Figure 1.

Table 3: Potential Energy Releases for 1 m$^3$ of stressed, gassy coal. Note no account is taken of energy consumed in the failure process

<table>
<thead>
<tr>
<th>PROPERTIES</th>
<th>UNITS</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus</td>
<td>GPa</td>
<td>2</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td></td>
<td>0.3</td>
</tr>
<tr>
<td>UCS</td>
<td>MPa</td>
<td>12</td>
</tr>
<tr>
<td>Mean stress</td>
<td>MPa</td>
<td>12</td>
</tr>
<tr>
<td>Sorption Pressure</td>
<td>MPa</td>
<td>4</td>
</tr>
<tr>
<td>Gas content</td>
<td>m$^3$/t</td>
<td>14.8</td>
</tr>
<tr>
<td>Strain Energy</td>
<td>MJ/m$^3$</td>
<td>0.16</td>
</tr>
<tr>
<td>Pore gas expansion</td>
<td>MJ/m$^3$</td>
<td>0.03</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Diffusion Coefficient</th>
<th>m$^2$/s</th>
<th>1x10$^{-8}$</th>
<th>1x10$^{-10}$</th>
<th>Particle Size mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diffusional Energy</td>
<td>MJ/m$^3$</td>
<td>0.94</td>
<td>0.47</td>
<td>0.1</td>
</tr>
<tr>
<td>Diffusional Energy</td>
<td>MJ/m$^3$</td>
<td>0.47</td>
<td>0.009</td>
<td>1</td>
</tr>
<tr>
<td>Diffusional Energy</td>
<td>MJ/m$^3$</td>
<td>0.009</td>
<td>0.001</td>
<td>10</td>
</tr>
</tbody>
</table>

| Energy in Falling 1 m     | MJ/m$^3$| 0.014       |

If failure does not take place then an outburst cannot. Thus determining whether failure will occur is an important part of the process of determination as to whether an outburst will take place. For failure to lead to an outburst, fragmentation must also take place. This fragmentation appears to be controlled by pre-existing structure, either in the form of already sheared coal or coal that will fragment. Thus determination of the structure is vitally important. This can be arrived at by examination of borecore by the process shown in Figure 9 and by use of the Pop Gun Test. Such examination and tests can however only be undertaken where core can be retrieved. If the coal is too sheared or fragments on core drilling then collecting cuttings from open hole drilling is an alternative. These may then be examined for particle size distribution. Despite the resistance within Australia this is best undertaken using air flush drilling as it provides virtually instant cuttings retrieval and therefore good sample location (Gray, 2011). Air flush is the only mode of drilling that is successful in some highly disturbed coals such as those in the Karaganda Basin of Kazakhstan and does not produce ignitions.
The next test of whether a coal will fail is its toughness. The Protodyakanov Index and q Index tests are measures of toughness. They too are dependent on the structure of coal on the scale that is tested. These simple tests form the basis for both Russian and Chinese determination as to whether a coal will outburst or not. In the Chinese case they are used in combination with a coal structural definition a measure of gas pressure and an adsorption rate index in the determination of outburst conditions. In the case of mines in the Kuzbass an equation linking pressure and toughness as determined by the Protodyakanov Index is used.

If fragmenting failure is likely to take place then the important question is what is the acceptable level of energy release that is associated with it? This energy release can be estimated by the processes used to arrive at Table 3 and detailed by Wood and Gray (2015). This should be estimated on the basis of energy per unit volume and expected energy release of the entire failed mass. Thus the total effect of an outburst from large faulted zones can be estimated. Given the apparently low transfer of potential energy into kinetic energy during the outburst process it would appear that a value of 0.1 MJ/m³ is a realistic initial threshold value of potential energy for a medium sized outbursting volume (100 m³) yielding a total potential energy of 10 MJ. However 10 MJ still represents a lot of energy if it were by the nature of the outburst process to be delivered in its entirety to the entrained particles. If the structure is expected to be larger than 100 m³ then the allowable energy per unit volume should probably be reduced to a lower value. The degree to which it should be reduced is uncertain and deserves further attention as indeed does the 0.1 MJ/m³ threshold.

The process of outburst threshold determination is seen as:

1) Determining whether the coal is already in broken form in the ground.
2) Determining whether the coal will fail (contains sufficient structure that it will fragment) under mining conditions
3) In the event of failure by either method then estimating the potential energy that may be liberated.

Determining that the coal has failed or may fail can be achieved by the inability to retrieve intact core. It may also be possible to determine it by the return of more finely broken cuttings than would normally be the case with open hole drilling. The measurement of an excessively large volume of cuttings from open hole drilling is also an indicator that this is the case. The determination of whether coal will fragment and its likely sizing appears to be able to be determined by the examination of core or coal lump in polished section as shown in Figure 9. This may also be confirmed by the use of the Pop Gun Test. In the latter case a pressure of fragmentation may be determined and a good safety margin allowed below this for safe mining. Once it has been decided that failure will take place then the focus should be on what can be done to limit the potential energy release. One option is gas drainage to a level where the expected coal fragment size would not have enough energy to pose a serious problem. The other is to de-stress the seam to be mined which will also achieve the release of gas. This is the common process in Eastern Europe, Russia and China. A seam is chosen that is less prone to outbursting usually because it is tough and has less structure. It is mined and then the adjacent seams are de-stressed and de-gas through drainage of the relaxed structure. These can then be safely mined.

CONCLUSIONS

This paper describes the some of the approaches used to outburst risk determination worldwide. These take into account gas pressure (by preference to gas content), desorption rate, coal toughness and structure. The Australian approach is by comparison extremely limited, taking into account only gas content or a limited variant of it in the form of the DRI index introduced by GeoGAS Pty Ltd. This approach is inadequate and fails to take into account the other factors that contribute to outbursting.

It is quite possible to mine coals at higher gas content thresholds than are currently permitted in Australia provided that the other contributory factors to outbursting are not present. This has very
significant consequences for improving mine productivity. Conversely where such factors as fine gouge material exist in a faulted zone it is quite possible that the gas content part of the threshold parameters may need to be lowered below currently in use.

The determination of outburst risk should therefore be revised to take account of these additional factors. This could be undertaken in a similar basis to that used overseas, namely taking into account the structural geology in combination with the results of several measured parameters either used separately or combined in some equation.

The alternative revision is to determine the conditions required for failure and to apply a good margin of safety to these. If failure is not considered to be likely then mining may proceed. If however failure is considered to be a possibility then the approach needs to change to one of determining what the potential energy release may be. If this energy release is too high, taking into account the volume of the likely failure, then measures must be adopted to reduce the energy release. These include gas drainage and stress relief techniques.

If either of the alternative approaches are adopted it will mean more measurements and in particular a determination of changing coal structure. This will require a different level of alertness by operators, especially in their drilling operations and the measurement process associated with them. A failure to incorporate the determination of changes in coal structure would automatically lead to an assumption of poor conditions and therefore the need to change the threshold values of the other contributing factors to outbursts.

The option is to maintain the status quo based on gas content measurement. This has generally been successful in preventing outbursts because of the rigour by which it is applied. This is enabled by the very simple nature of the process. It has however cost Australian mining very dearly in terms of lost productivity.

ACKNOWLEDGMENTS

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DETERMINATION OF THE GAS CONTENT OF COAL

Abouna Saghafi¹,²

ABSTRACT: In coal mining the gas content of coal is required primarily to quantify the gassiness of coal for safe mining, but also to quantify potential greenhouse gas emissions from mining. In Australia the gas content of coal is determined using a direct method, whereby the gas desorbed from solid or crushed coal is collected and the volume and composition of the desorbed gas are measured. The determination of gas content is associated with errors of measurement of the volume and composition of the gas. It is undertaken at several stages of gas desorption. Relative errors and resulting uncertainties of determination are more significant for the estimation of lost gas during drilling and gas remaining in coal following the completion of the standard stages of measurements, whence the rate of gas desorption is significantly reduced. This paper discusses the current Australian method and potential errors and uncertainty associated with this method. A new method of measurement for measurement of remaining gas in coal following the completion of standard gas content testing is also suggested. The new method should allow the release of almost all remaining gas in powdered coal following the last stage of standard gas content testing.

INTRODUCTION

Gas in coal occurs in both adsorbed and free phases. The adsorbed gas, which constitutes almost all desorbed gas from coal, is in a liquid-like state and covers mostly the internal meso- and micro-pores surfaces. The free phase fills the void volume of all pores in coal. The free and adsorbed phases are in equilibrium which is expressed mathematically through relationships between the amount of gas adsorbed and the pressure of the free phase adjacent to the adsorbed layer. These relationships are either derived from an equilibrium kinetic approach to gas adsorption, assuming a monolayer formation mechanism (e.g. Langmuir 1918) or are derived from a potential theory, assuming a pore filling mechanism (e.g. Dubinin 1968).

The gas content of coal refers to the volume of the adsorbed phase in coal as the released gas from coal during gas content testing consists mainly of desorbed gas from the adsorbed phase. The contribution of the free gas phase to the total gas volume in coal is generally small (<10%) for shallow coals (<500 m), however it increases with depth and should be considered at great. Methods of measurement of gas content of coal used in coal producing countries are generally variants of the methods older methods developed in European countries (see Firedamp Drainage, Handbook for the Coalmining Industry 1980). In most methods gas is initially allowed to desorb ‘slowly’ from solid coal and the volume of the desorbed gas with time is measured. Such curves can be useful to estimate desorption parameters of coal such that a ‘characteristic desorption time’, otherwise known as τ parameter, is obtained. In early methods the slow desorption is monitored until no ‘measureable’ desorption takes place (see Kissell et al., 1973; Diamond and Levine 1981 and Diamond and Schatzel 1998). In subsequent methods the slow desorption period is shortened and coal sample is crushed and pulverised (generally to less than 200 µm) to desorb its gas rapidly. Variants of later methods have been used as early as 1960’s in Europe (Bertard et al., 1970). Although various methods are used in coal producing countries today, at present there are only limited number of agreed standards across the world for the determination of gas content of coal. In Australia, since the early 1990’s work has been conducted to produce standard ‘guides’ for the determination of gas content. The most recent document, which was developed under the auspice of the Australian Standards, is AS 3980-1999 (1999), which is an update and improvement of a previous document (AS 3980-1991 - 1991). More recently the US ASTM also published a document (D7569-10 -
2010) on the determination of gas content with heavy reference to the Australian AS 3980 document. Currently a new standard is being developed under the auspice of the Australian Standards, expected to be available in the course of the current year.

In the next sections of this paper first the current Australian method of gas content testing is briefly described and aspects of uncertainties in gas content determination and generic approaches to estimate these uncertainties are presented. A method of direct measurement of gas volume (remaining gas content) in powdered coal is also presented.

**Determination of the gas content of coal – Australian Method**

A variant of the fast desorption method, otherwise known as quick crash method, was developed in Australia and used in the early 1990’s (for example see Williams et al., 1992). The method was then extensively used and is the basis of the current guide (AS-3980 1999). It has been validated through repeatability and reproducibility experiments and through inter-laboratory round robins exercises (Saghafi et al., 1998; Danell et al., 2003 and Saghafi 2012). This current Australian gas content guide (Standards Australia AS-3980, 1999) describes three main stages for the determination of the gas content of coal. Each stage of gas content testing provides a volume of gas corresponding to that stage. These volumes are represented by $Q_1$, $Q_2$ and $Q_3$ parameters and their sum is presented by $Q_m$ or the 'measured gas content' (AS-3980, 1999):

$$Q_m = Q_1 + Q_2 + Q_3$$

(1)

The first component of measured gas content is $Q_1$, which is the amount of gas lost from a coal sample during drilling, during its retrieval from the borehole, and before sealing in a desorption canister. Lost gas is not directly measured but estimated from the measured gas desorption rate once coal is sealed in the desorption canister.

The second component of measured gas content is $Q_2$, which is the amount of 'slow' desorbed gas from the intact (non-pulverised) coal sample sealed in the desorption canister during its transport to the laboratory and prior to its pulverising at the third stage of gas content testing. In the traditional method of testing called the 'slow desorption method', desorption measurement is conducted until the rate of gas desorption is so small that is not sensed by the measuring device. In the standard method, or 'fast desorption method', however, this stage can be very short and is independent of the rate of desorption at completion of this stage.

The third and last component of measured gas content is $Q_3$, which is the amount of 'fast' desorbed gas from the pulverised coal. This component is measured by taking sub-samples from the main coal sample and crushing them to a fine powder using a crusher. Note that gas desorption measurements should be carried out at near atmospheric gas pressures (partial pressure of gas in the crusher bowl should be about atmospheric).

Gas may remain in coal beyond the three stages of gas content testing. This ‘remaining’ gas, represented by $Q_3'$, can be determined directly by measurement or indirectly using the adsorption properties of coal. However, its determination is not yet considered by the standard method for gas content testing in Standards Australia AS-3980 guide.

If $Q_3'$ is measured then the total gas content, or $Q_t$ can be calculated. It is the sum of the 'measured' and 'remaining' gas contents:

$$Q_t = Q_m + Q_3'$$

(2)
Figure 1 shows the relationship between the various gas content components and coal gas adsorption properties (adsorption isotherm). The free partial gas pressure in pores should reduce to a certain level to allow the release of the adsorbed gas from coal.

![Graph showing the relationship between gas content and adsorbed gas](image)

**Figure 1: Relationship between the gas adsorption isotherm and the components of gas content, not to scale**

Note that all components of gas content are expressed in terms of gas volume per unit mass of coal (m$^3$ per tonne), where volumes are presented for standard temperature and pressure (STP) conditions. For the Australian standard (Australian Standards AS-3980, 1999), the STP conditions are 101.325 kPa and 20°C. Note that ASTM D7569-10 (2010) recommends using 101.3 kPa and 15°C for gas volume calculation so to conform to API standards.

Measurement of the desorbed gas volume from coal (during all three stages) is carried out using water displacement in an inversed cylinder held above a water basin (Figure 2). Prior to measurement a level of suction (vacuum) is kept in the head space of the cylinder so that a water column in the cylinder is formed above water level in the basin. Desorbed gas is allowed to flow into the head space, which displaces water down and reduces the water column height, so that the volume of desorbed gas can be measured. To reduce the dissolution of desorbed gas (particularly CO$_2$) in water, acidified water is used and desorbed gas enters the measuring cylinder from the top (Figure 2).

**Uncertainties associated with the determination of gas content**

Only limited studies have been undertaken to evaluate the uncertainty of this method for the determination of gas content. Saghafi et al (1998) studied the variation (reproducibility) of measuring the gas content of coal by comparing results from three Australian laboratories, and found that there was ±15% variation in the values of gas content for a suites of similar coal samples measured concurrently by these laboratories. The samples were produced from the same cores obtained from in-seam drilling. Coal core pieces were mixed to produce similar sub-samples for participating laboratories.
The uncertainty of gas content determination within a single laboratory depends upon field and laboratory equipment and measuring tools used but also on operators’ skills and judgement. For example the type of graduation of the measuring cylinder and the material it is made of, i.e. whether it is made of glass or plastic, can all affect the measurement error and the level of uncertainty of gas content determination. Operator judgement influences the final uncertainty. For example, for the determination of $Q_1$ it is required that the operator evaluates the rate of initial desorption from measurements of the desorption curve in the field and time zero, that is a point in time when the coal started to release its gas in the exploration borehole. Different operators may have different judgement on the value of parameters of desorption and hence end up estimating different values for $Q_1$. Similar concerns are also raised for $Q_2$ and $Q_3$ and when one should decide that $Q_3$ is completed and how much gas may be still in coal.

In the next section some of the uncertainties associated with estimation of $Q_1$ and measurement of $Q_2$ and $Q_3$ are discussed.

**ESTIMATION OF LOST GAS ($Q_1$) AND ASSOCIATED UNCERTAINTIES**

Gas is released from coal as soon as the water pressure falls to below the combination of capillary and free gas pressures in the coal pores. Lost gas is the gas which is released in the borehole and following the retrieval of coal at the surface prior to sealing the sample in a desorption canister. This gas is lost because as it can not be measured. However, the amount of lost gas may be estimated assuming that desorption follows a certain kinetic which can be established from an initial gas desorption curve from coal sample in the canister (accumulated volume of desorbed gas is measured against time).

In estimating the lost gas, one important input parameter is time zero or the time when coal starts to release its gas in the borehole. In AS-3980 (1999) time zero is defined as the time mid-way between the start and completion of coring for the section to be sampled. However, this can be subject to discussion as the length of the water column and piezometric pressure can significantly influence the onset of desorption. The other parameter in question is the temperature, which changes as the sample is retrieved from the borehole. The elapsed time since time zero, can be called ‘lost time’ and in this paper it is expressed by the variable $t_e$. The temperature of drilling fluid may be used as the desorption
temperature. This aspect has not been commented on AS-3890, and to opinion of the author should not be considered a major source of error.

**Initial rate of desorption as basis for estimation of Q₁**

Experimental data suggest that gas desorption from coal follows a first order diffusion equation and hence the evolution of gas desorption in a canister can be explained in terms of diffusion physics. In this regard, the cumulated volume of gas released from coal \((q)\) over a period of time is proportional to the square root of the product of effective diffusivity \((D_e)\) and the elapsed time \((t)\) since the start of desorption or the start of the measurement of desorption:

\[
q(t, D_e) = \alpha \sqrt{D_e t}
\]  

(3)

If the diffusivity of gas in coal remain constant for the period of measurement then the cumulative volume of gas will be proportion to square root of time only and the volume \(q\) is a linear function of square root of time. This is what is observed during field measurement of gas desorption which show that for short period of time the hypothesis of the linearity of the volume of gas against the square root of time is held. Assuming that the effective diffusivity of gas in coal has not changed since the start of desorption in the borehole and the desorption measurement in the canister then the lost gas can be estimated by back extrapolation of this line. If after a time \(t_e\) (lost time) coal is sealed in the desorption canister and the measurement of the volume of desorbed gas \((q)\) is plotted against square root of time \(t\), according to Eq. 3, the shape of curve should follow the following equation:

\[
q(t) = k\sqrt{t} - Q_1 = k(\sqrt{t} - \sqrt{t_e})
\]  

(4)

Note that the origin for the time axis in Eq. 4 is the time when coal started to release its gas in the borehole (not the time when the coal is sealed in the canister and measurement starts). The slope of the regression line, or coefficient of proportionality is \(k = \alpha D_e\). This latter is determined from the data from the initial desorption curve measured in the field. Coefficient \(k\) depends on environmental conditions, mainly temperature of coal, but also on the initial gas content and diffusion properties of coal.

**Uncertainty of determination of Q₁**

Besides the error associated with measurement equipment, two main parameters affecting the results are the value of lost time \(t_e\), and the slope of initial desorption line \(k\) in Eq. 4.

The assumption of linearity of the cumulated volume of desorbed gas with the square root of time and the back extrapolation of the regression line to estimate the lost gas is only valid for short values of \(t_e\) and error of estimation increases with larger value of \(t_e\). Hence, the lost time shall be kept as short as the operation would allow. Evaluation of the true \(t_e\) is difficult and can only be based on assumption. The regression line \((k)\) can vary because of environmental conditions; with coal drying at the surface \(k\) increases. Similarly an increase (or decrease) in coal temperature at the surface can increase (or decrease) the value of \(k\). This often happens in the field after coal is placed in the canister. It takes some time for coal to reach the assumed in-situ (pre-set) temperature of the water bath in which the coal canister is placed.

Figure 3 shows an example of the measurement of initial rate for the determination of \(Q₁\). In this example we assume that the value of the lost time \((t_e= 20\) minutes) is reasonably correct. However, for the slope of gas desorption curve, we have two values \((k₁ = 0.22\) and \(k₂ = 0.50\) m³/t per min\(^{0.5}\)). It could be asked how the operator should evaluate the data and then work out a most appropriate value for \(k\) to determine lost gas volume? One reasoning could be that gas desorption has a lower rate at the start of desorption because coal has not reached the pre-set temperature (in-situ temperature) in the water
bath. Once coal reaches its pre-set temperature then desorption should take place at in-situ temperature. Operator may then choose the slope \((k_3)\) for calculation of \(Q_1\), which yields an estimate of \(Q_1 = 2.2 \text{ m}^3/t\) for lost gas.

Note that \(t_e\) and \(k\) are two independent variables for the estimation of \(Q_1\) and the choice for value of \(t_e\) does not affect the choice for value of \(k\). Therefore the calculation of \(Q_1\) is only affected by different rates of desorption (slopes of desorption lines).

![Figure 3 Determination of lost gas \(Q_1\) and changes in the initial canister gas desorption rate](image)

**A generic method for the calculation of the uncertainty of \(Q_1\)**

Various estimation error affect the level of uncertainty of \(Q_1\). Besides the errors associated with equipment and measurement tools (such as the thermometer for measuring the temperature or the measurement of the volume of gas using a measuring cylinder) there are larger uncertainties associated with accurate evaluation of the lost time and the rate of initial desorption \((t_e\) and \(k\) in Eq. 4). If the error of evaluation of these variables is expressed in terms of partial derivatives, the variation in the value of \(Q_1\) due to the variation in evaluation of values of \(t_e\) and \(k\) is:

\[
\delta Q_1 = \frac{\partial Q_1}{\partial t_e} \delta t_e + \frac{\partial Q_1}{\partial k} \delta k
\]

Using partial derivation of Eq. 4 and assumption of quadratic additions of individual uncertainties (see for example JCGM, 2008), the uncertainty of \(Q_1\) associated with uncertainties of lost time and the rate of desorption is:

\[
\delta Q_1 = \pm \sqrt{\frac{k^2}{4t_e} (\delta t_e)^2 + t_e (\delta k)^2}
\]

For example for the estimation of \(Q_1\) in Figure 3, assuming that the lost time and desorption rates are expressed as \(t_e \pm \delta t_e = 20 \pm 4\) minutes, and \(k \pm \delta k = 0.5 \pm 0.1 \text{ m}^3/t\) per min\(^{0.5}\), respectively, then the uncertainty of estimation of lost gas is \(0.5 \text{ m}^3/t\) and the lost gas is \(Q_1 = 2.2 \pm 0.5 \text{ m}^3/t\).
MEASUREMENT OF SLOW ($Q_2$) AND FAST DESROBED GAS ($Q_3$) AND ASSOCIATED UNCERTAINTIES

Once coal is sealed in a desorption canister after its retrieval from borehole any gas desorbed from coal is part of the $Q_2$ stage of gas content testing including field gas desorption for estimation of $Q_1$. Slow desorption may be left to continue until the operator decide that the sample is ready to go through the $Q_3$ stage of measurement and fast desorption by crushing the sample either in the same canister or in a separate crusher.

Measurement of $Q_2$ is generally straight forward and if the system is gas tight and properly sealed then, the uncertainty of measurement would be mainly related to the uncertainty in evaluating the coal mass, the void volume in the system and the measurement of pressure and temperature. Measurement of the volume of gas released for both $Q_2$ and $Q_3$ stages of gas content testing are generally measured using the same technique (water displacement as shown in Figure 2). The total desorbed gas in stage $Q_2$ is determined by adding successive volume increments (water displacements) over the period of measurement:

$$Q_2 = \frac{1}{m} \sum_{i=1}^{n} \Delta V_i$$

(7)

where $m$ is the mass of sample, and

$$\Delta V_i = V_i - V_{i-1}$$

(8)

Volumes $V_i$ and $V_{i-1}$ are the volume of gas occupying the system void at two consecutive steps of $i-1$ and $i$ of the measurement. The volume of gas at STP conditions at step $i$ is:

$$V_i = \frac{P_a}{P_{a,i}} \left( V_{\text{void}} + V_{\text{cy},i} \right) T_{a,i}$$

(9)

where $P_a$ and $T_a$ are STP absolute pressure and temperature (°K) and $T_{\text{void},i}$ and $T_{\text{cy},i}$ are absolute temperatures in the system (void volume in the desorption canister/crusher and tubing) and in measuring cylinder at step $i$ of measurement of gas volume. $V_{\text{void}}$ is the void volume in the system (void volume in coal canister and tubing and fittings) and $V_{\text{cy}}$ and $V_{\text{cy},i}$ are the full volume capacity of measuring cylinder and empty volume of this cylinder at step $i$ of measurement. Pressure $p_i$ is the absolute pressure of gas in the system. Note that this pressure is smaller than atmospheric pressure due to the suction exerted by the water column (Figure 2). For instance if at step $i$ the height of water column is $h_i$ then pressure $p_i$ is:

$$p_i = P_a - ah_i$$

(10)

Note that if pressure is measured in kPa and height in metre then $a = 9.8$. The height of the water column $h_i$ can be measured directly or estimated from the following equation if the graduation of the cylinder are in unit of volume (Figure 2):

$$h_i = (1 - \frac{V_{\text{cy},i}}{V_{\text{cy}}})h_0$$

(11)

where $h_0$ is the height of the water column at the start of measurement (when measuring cylinder is fully filled with water) and $h_i$ is the water column height at current step $i$ of measurement of water displacement in the cylinder. The two volumes $V_{\text{cy}}$ and $V_{\text{cy},i}$ are total volume of the measuring cylinder and its void volume at step $i$ of measurement, respectively (Figure 2).

For the $Q_3$ stage of gas content testing, the same calculation can be carried out as the same method of measurement for the volume of the desorbed gas is used. However, the results are also affected by the
partial pressure of the seam gas in the system as well as temperature of the coal (which can become quite high during crushing).

Another factor is the final particle size distribution of powdered coal. It is recommended to keep temperature and pressure at near STP conditions if possible and that the particle size be below 200 µm. Smaller size have larger exposed surface and most desorbable gas would be released during crushing. The partial pressure of gas near atmospheric pressure would produce the 'measured gas content' value near or equal to the desorbable gas content value of coal.

**Uncertainty of measurement of Q₂ and Q₃: effect of equipment used and void volume**

Many factors affect the volume of gas desorbed during Q₂ and Q₃ measurements. Some factors have effect on the physics of desorption and increases or decreases the amount of gas that can be released from coal; factors such as the moisture content of coal and excess water vapour in the system or the composition of air and gas in the void space and partial pressure of seam gas (whether the canister/crusher is rinsed and filled with an inert gas before measurement) can have significant effects on desorbable gas from coal. Another aspect of uncertainty is related to the equipment used such as the type and accuracy of the measuring cylinder or the remaining volume of void in the system once the coal sample has been placed into the desorption canister. In this section a simple method is presented for the calculation of the uncertainty of gas content due to the uncertainty of the measured gas volume, which involves the void volume in the system and in the measuring cylinder.

For the sake of simplicity assume that the temperature is kept the same across the measurement system (i.e. \( T_{\text{void},i} = T_{\text{cy},i} = T_i \)), so that Eq. 9 could be simplified to:

\[
V_i = \frac{T_i}{P_i T_i} (V_{\text{void}} + V_{\text{cy},i}) P_i
\]

The uncertainty of the volume of desorbed gas using Eq 12 depends on the uncertainties associated with values of temperature (\( T_i \)) and \( p_i \), which change in each measurement step. It also depends on the uncertainty of the void volume in the system, and the uncertainty of the volume of empty space in the measuring cylinder (\( V_{\text{cy},i} \)).

Note that \( p_i \) is not an independent variable but is dependent on \( V_{\text{cy},i} \), hence for the uncertainty of determination of the volume of desorbed gas associated with \( V_{\text{cy},i} \) and \( p_i \), the effect of the latter can be evaluated in terms of the uncertainty in the measurement of \( V_{\text{cy},i} \).

If we use a partial derivation technique and assumption of quadratic additions of individual uncertainties, then the variation in the calculated volume of desorbed gas in the system would be a function of the uncertainties in data on the void volume in the system (\( \delta V_{\text{void}} \)) and the void volume in the measuring cylinder (\( \delta V_{\text{cy},i} \)) as follows:

\[
\delta V_i = \pm \frac{T_i P_i}{P_i T_i} \sqrt{(\delta V_{\text{void}})^2 + (\delta V_{\text{cy},i})^2}
\]

(13)

Note that the uncertainty due to volume of the change in pressure (\( \delta p_i \)) is ignored in developing the above equation (Eq. 13). The uncertainty in volume of desorbed gas calculated in Eq. 13 is associated solely with the uncertainty of evaluating the void volume in the system and the uncertainty in reading the amount of water displacement in the measuring cylinder.
DETERMINATION OF THE REMAINING GAS IN COAL ($Q'_3$)

Once $Q_3$ measurement is completed there is still a chance that volumes of gas remain in coal. Depending on the purpose of gas content testing, the determination of the remaining gas volume may be required (e.g. for the estimation of greenhouse gas emissions from mining). The amount of $Q'_3$ (remaining gas) left in coal depends on a multitude of factors such as seam gas partial pressure in the system, moisture and diffusivity properties of the coal. Large volumes of gas can also be retained because of wettability properties of coal (contact angle of gas and water interface with coal surface). It is often suggested to determine $Q'_3$ indirectly using the adsorption isotherm (Figure 1) and the final partial pressure of the seam gas in the $Q_3$ measuring apparatus. This implies that gas partial pressure in the $Q_3$ apparatus be determined and an isotherm of powdered coal be measured following the $Q_3$ measurement. However, this indirect method for determining $Q'_3$ only relies on the partial pressure and the isotherm. The measurement of these parameters would introduce additional uncertainties.

Direct method of measurement of $Q'_3$

Over the last few years we have developed a direct method for measuring $Q'_3$ (Saghafi, 2011; Saghafi, 2012). In this direct method the crushed coal in the $Q_3$ apparatus is vacuumed and then flushed with an inert gas (e.g. $N_2$, He or Ar depending on the GC carrier gas). Coal is then left to desorb its gas over a period of time. For most coals gas desorption can be more complete and enhanced if the apparatus is pressurized above atmospheric pressure. Once the pressure and temperature in the apparatus reach their equilibrium, gas pressure and temperature are recorded. Gas samples are also collected from the apparatus and measured for the desorbed gas composition (Figure 4). Using data on gas concentration of seam gas in the system ($c$, vol/vol), total pressure ($p$) and temperature of the gas in the apparatus $Q'_3$ can be calculated as:

$$Q'_3 = \frac{1}{m} \frac{T}{P_a} c V_{\text{void}} \frac{p}{T}$$

(14)

Figure 4: Measurement of the remaining gas ($Q'_3$) using a direct method; gas is allowed to release from pulverised coal over a period of time into an inert gas atmosphere (modified from Saghafi, 2011)

CONCLUSIONS

Gas content is the most important parameter for characterising a coal seam in relation to gas emissions from mining. In this regard the current Australian method for the determination of gas content requires revision to reduce the uncertainty and increase the accuracy of the method. In light of the new understanding of the interaction between gas and coal and the adsorption and desorption mechanisms,
enhancement can be made to the current method and new approaches are required to increase the accuracy and measurability of gas content. Methods for the determination of uncertainties of gas content using the current method should also be part of the new methods for determination. Inventories on gas emissions require total gas in coal to be determined. Based on these new requirements, particularly in relation to greenhouse gas emissions, current methods for the direct measurement of remaining gas in coal should be further developed.

REFERENCES


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THE EFFECT OF AIR COLUMN IN TRANSPORT CANISTERS ON MEASURED GAS CONTENTS

Naj Aziz and Ali Mirza

ABSTRACT: Canister desorption is a widely used technique to measure the gas content of coal. The gas content data when normalised to volume/weight and multiplied by coal seam mass is used to estimate the gas in place in an area around the cored hole. However, the gas content and its components are likely to be influenced by trapped air in the canister at the time of the coal enclosure and subsequent sealing. Freshly cored coal samples were collected from three mines, mining coal from the Bulli seam, Sydney Basin. The underground mines were Appin West, West Cliff and Tahmoor. The research programme spanning a period of four years, focused only on the influence of the entrapped air in the canister on coal gas percentage of each constituent. It was found that the percentage of each coal gas constituent was influenced by the trapped air in the canister space. The effect of trapped air was extended to the component percentage of the gases in the crushed coal samples, normally used for the estimation of Q3.

INTRODUCTION

Canister desorption is a widely used technique to measure the gas content of coal. The gas content data when normalised to volume/weight and multiplied by coal seam mass is used to estimate the gas in place in an area around the cored hole. However, the gas content and its components are likely to be influenced by the presence of trapped air column in the canister, particularly when the canister is not fully occupied by the coal column and the trapped air, if not removed, may likely to be a concern.

Gas composition has been widely used to mean the percentage of each gas in a mixture of gas liberated from solid intact coal. This is not an appropriate phrase to be used when dealing with gas content measurement using canisters. The use of the phrase “gas component percentage”, is an acceptable terminology as it refers to the percentage of each gas in the canister space, which may be influenced by the trapped contaminant air.

Three systems of countering the influence of air column on the gas content of coal have been recognised in some gas processing laboratories are (a) by neutralising the canister air by flushing the air with helium gas; b) by calculating the level of air components in the canister and then recalculate the gas content of the gas in the canister, and c) by filling the air space with inert solid blocks and flushing out the rest of the remaining air with inert gas. Polymeric solid blocks rods are known to be used for canister’s unoccupied space filling, as these rods do not react with coal gases. No consideration has been given in this paper on the issue of water vapour correction as the objective of the paper was primarily for coal gas component percentages analysis and its variations with trapped air contaminants.

While all these techniques are used, however, no studies have previously been reported to scientifically examine the impact of the air trapped in canisters on the overall estimation of (a) the gas content of coal (Qm), where Qm = Q1 + Q2 + Q3, (b) coal gas component percentages, (c) component percentages of both Q2 and Q3. This paper reports on the findings of the study, and demonstrates the importance of eliminating or flushing out air pockets in canister columns.

MEASUREMENT OF GAS CONTENT OF COAL

Measurement of the gas content of coal samples involves three stages of Standards Australia (1999),

(i) Determining the gas lost from the coal sample during core sample recovery (Q1),
(ii) Measuring the gas desorbed from the coal sample, while sealed in a desorption canister (Q2),
(iii) Measuring the gas released from a coal sub-sample during crushing (Q3).

The gas content measured during the above listed stages is added together to give the total measured gas content (Qm) as:

\[ Qm = Q1 + Q2 + Q3 \]

Qm represents the total volume of gas released per unit mass of coal when the ambient gas partial pressure is maintained at one atmosphere. Given the potential for variable temperature and atmospheric pressure conditions during gas content measurement and differences in the mineral matter content of the coal samples, the results are typically normalised with Qm being reported in NTP (20°C and 101.325 kPa) and 10% non-coal matter (NCM) (Close and Erwin 1989 and AS, 3981–1999).

Two desorption methods are used in Australia, the fast and slow desorption methods, to directly measure the gas content of coal samples, as described in Australian Standard AS3980 (1999). Q1 is an estimated quantity of the gas component in coal; it is generally accepted to be the least accurate component of Qm (Mavor et al., 1992; Diamond and Schatzel 1998; Diamond et al., 2001). The desorbed gas component (Q2) is a measure of the volume of gas released from a coal sample whilst contained in a desorption canister. The duration of the Q2 test may be short in the case of a fast desorption method less than one day or much longer in the case of slow desorption testing, not less than five days.

The crushed gas component (Q3) is a measure of the gas liberated from a coal sample following crushing. Following completion of the desorbed gas test the coal core is removed and a representative sub-sample collected and sealed into a crushing or grinding mill. Following crushing the volume of gas liberated from the coal sample is measured using a water column similar to that used in the desorbed gas measurement.

In fast desorption testing, where the desorption time is less than five days and typically less than one day, Q3 represent a large percentage of Qm. In slow desorption time lasting over a period of time equal to or greater than five days, Q3 is quite low, and represent the residual gas content of the sample. Residual gas content is the volume of gas per unit mass of coal that is naturally retained within the coal and not readily released from an intact sample. The residual gas content also represents the portion of Qm that will not be liberated into the mine atmosphere from mined or intact coal (Diamond and Schatzel, 1998).

Residual gas content is also an important consideration in the evaluation of coalbed methane gas recovery potential as it represents the portion of Qm that will not readily flow to gas drainage boreholes (Diamond and Schatzel 1998). Gas Chromatography (GC) is used to analyse the gas component percentages in coal.

**EXPERIMENTAL STUDY**

The influence of the trapped contaminant air in the canister on gas component percentages of a coal column contained was studied experimentally. Fixed lengths of freshly drilled coal core samples were inserted into different length canisters. Two sets of three canisters 350 mm, 800 mm and 1000 mm in length were used, which allowed the analysis of samples with reduced air columns contained in their respective canisters. No attempt was made to determine the true gas content of coal in this particular study, as the objective was to examine the changes in gas mixture component percentage caused by the presence of mine air trapped in the canister. The procedure adopted consisted of:

1) Studying the effect of varying air columns in transport canisters on the coal gas component percentage, and
2) Elimination of the presence of trapped air in canisters by either flushing out the trapped air with inert helium gas, or filling the unoccupied canister space with polymer rods plus helium gas to completely expel air out of the transport canister.

Freshly cored coal samples, 45 mm in diameter were obtained from three mines, mining coal from the Bulli Seam, of the Sydney Basin. The mines were Tahmoor, Appin West and West Cliff. The 300 mm long coal samples, once cored were inserted and sealed in different length canisters. The loaded canisters were taken to the gas laboratories of both UOW and Illawarra Coal – South 32 at Cordeaux Colliery respectively. All three mines are known to have high levels of mine gases, with varied mixture gas component percentages, ranging from 90% methane to 90% carbon dioxide (Lama and Bodziony, 1996). The duration of the study was prolonged to over four years, thus the sample collection for this study was varied over that period of the study.

The location of cores collected from three mines were:

**Tahmoor Mine**: Coal Core samples were collected from Panel 810, maingate A at about 5 m away from the intersection between 47 and 48 as shown in Figure 1a. Three canisters were used for sample collection with the shortest canister having internal space length of 350 mm, 55 mm ID.

**West Cliff Mine**: Core collections occurred in two different time frames, in 2011 and later in 2014. The samples collected in 2014 were drilled in Panel 516-38.5-1/1 shown in Figure 1b, the borehole direction was 45.5° making it perpendicular to the cleat. The samples were retrieved from the drilled hole at a depth of 350 m.

**Appin West Mine**: Four sets of samples were taken from Appin West Mine. Sample sets one and two were extracted from a drill hole located in the panel 705 maingate, 8 Cut Through (C/T). Sample set one was first drilled at a depth of 20 m before the hole was continued to a depth of 41 m and the core was excavated for sample set two. Samples for sample set three and four were also cut from 705 maingate, from A heading at 21 C/T. The direction of drilling was 15° to the cleat. For sample set three the cores were retrieved from a depth of 60.5 m, while set four samples were drilled from a depth of 80 m. The collection of samples from Appin West mine was confined to one period of samples collected in 2011.

In this paper field results are confined to two mines, West Cliff and Tahmoor, and additional tests were made in the laboratory on coal samples subjected to gas saturation using indirect absorption method (Lama and Bodziony 1996).
RESULTS AND DISCUSSIONS

Transport of coal in air trapped canister

Figure 2 shows three typical canisters used for collecting equal length coal core samples. Coal samples were collected in canisters and transported to the laboratory for gas mixture components percentage analysis. Samples of canister gas were extracted from each canister and fed to the GC. Three canisters of different lengths were used. They were 350 mm, 800 mm and 1100 mm long. All canisters contained trapped mine air when sealed. Changes in gas components were examined for both Q2 and Q3 of Qm.

![Figure 2: Coal transport canisters of varying length](image)

Following tests in each canister for changes in gas components percentages (canister gas composition) from extracted gasses released in the canister, coal samples were removed from canisters and crushed down to 200 μm. The released gas from crushed coal was collected and fed to the GC to determine the gas components percentages in Q3 stage. Table 1 lists the analysis results of the gas contained in three different length canisters from West Cliff Colliery, and both the Q2 and Q3 levels. Figures 3 and 4 show the bar charts for gas at both the Q2 and Q3 stages respectively. The following were found:

a) There was a clear variation in gas components percentage contained in different canister lengths,

b) The levels of oxygen and nitrogen were dependent on the quantity of trapped contaminant air in the canister,

c) The gas released from West Cliff Mine was rich in methane,
d) As expected the variation of the gas mixture component percentages was consistent with the gas content of the coal in the panel, where coal was cored. The level of CH4 recorded was higher in shortest canister with less trapped mine air. The methane component of the mixed gas reduced with increased air column. The changes in CO2 percentage were opposite to CH4.

Table 2 and Figures 5 and 6 show the results from Tahmoor Mine. Figures 7 shows the canisters with equal lengths of coal and Figure 8 show the procedure used for removing gas from the canister for GC analysis. The study found:

a) There were variations in gas components percentage in different canister lengths and canister air volume,
b) The level of O2 and N2 were higher in canisters with a large quantity of trapped air and in comparison with smaller shorter length canisters with relatively less trapped air,
c) Gas components percentage of coal from Tahmoor Mine was relatively rich in CO2, and poor in CH4.

Table 1: West Cliff Mine gas percentage components at both Q2 and Q3 Stages

<table>
<thead>
<tr>
<th>Gas</th>
<th>Q2 LAB (May 30-2014)</th>
<th>Q3 May 2015</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>350mm</td>
<td>800mm</td>
</tr>
<tr>
<td>CO2</td>
<td>4.73</td>
<td>3.79</td>
</tr>
<tr>
<td>N2</td>
<td>19.60</td>
<td>36.54</td>
</tr>
<tr>
<td>CH4</td>
<td>71.63</td>
<td>52.37</td>
</tr>
<tr>
<td>O2</td>
<td>4.00</td>
<td>6.37</td>
</tr>
</tbody>
</table>

Figure 3: West Cliff Mine Q2 stage gas component in percentage in coal in different length Canisters
Figure 4: West Cliff Mine Q3 component percentage analysis from different canisters

Table 2: Component percentage analysis for Q2 and Q3 - Tahmoor Mine

<table>
<thead>
<tr>
<th>Gas</th>
<th>Component percentage (% mol)</th>
<th>350mm</th>
<th>800mm</th>
<th>1000mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO2</td>
<td>0.34</td>
<td>13.35</td>
<td>23.03</td>
<td></td>
</tr>
<tr>
<td>N2</td>
<td>77.56</td>
<td>72.1</td>
<td>61.19</td>
<td></td>
</tr>
<tr>
<td>CH4</td>
<td>0</td>
<td>0.9</td>
<td>2.86</td>
<td></td>
</tr>
<tr>
<td>O2</td>
<td>21.21</td>
<td>12.8</td>
<td>12.21</td>
<td></td>
</tr>
</tbody>
</table>

Air Free canisters

In this part of the study, as no freshly drilled coal core samples were forthcoming from the local mines, it was decided to simulate gas components percentage study in the laboratory by the indirect absorption method. 50 mm diameter coal samples were cored out of freshly dug coal lumps brought to the laboratory from the mine. The cored coal samples were cut into 100 mm long samples and loaded individually into the sorption pressure vessel “bombs” and charged with methane gas to a pressure of 2000 kPa. The gas pressure was maintained constant until saturation. The gas saturated bombs were then opened and coal samples were readily transferred to two 800 mm long canisters (A and B canisters). Each canister was loaded with three 100 mm long core samples (total length of 300 mm) as shown in Figure 9. The first canister (Canister A) was sealed with normal atmospheric air trapped in the empty space. In the second canister (Canister B) 100 mm long 50 mm diameter polymer rods as shown in Figure 9b were inserted to fill the unoccupied space above the coal column. Canister B was next flushed with the gas to expel air and then sealed. Table 3 shows component percentages in two canisters with and without trapped air.
After two weeks, samples of gas were extracted from the canisters and analysed for gas component percentage. Figure 10 shows component percentage results from two canisters charged with CH4 gas. As can be seen from Table 3 and bar charts in Figure 10, there were variations between methane concentrations in two canisters, at both the Q2 and Q3 stages. This exercise demonstrated clearly that the trapped air in the canister unoccupied space has an influence on the gas component percentage similar to that obtained from cored coal samples directly drilled from the coal seam.
It is clear from all the tests reported in this paper that the changes in the gas mixture components concentration show that as the air column is increased there would be a decrease in both CH4 and CO2 percentages volume /volume (%v/v) ratio, while O2 and N2 would show an increase in %v/v ratio. These variations are clearly depicted in Tables 1 and 2 respectively at the Q2 percentage gas mixture component analysis stages. More significantly the air column in the canister will have an effect on the gas component percentages due to the much lower partial pressures of the seam gas in the canisters with trapped air column during desorption.
Table 3: Gas component percentage in two canisters with and without trapped air

<table>
<thead>
<tr>
<th>Sample A</th>
<th>%</th>
<th>Sample B</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>13</td>
<td>Oxygen</td>
<td>5.81</td>
</tr>
<tr>
<td>Nitrogen</td>
<td>61.7</td>
<td>Nitrogen</td>
<td>33.8</td>
</tr>
<tr>
<td>Methane</td>
<td>23.5</td>
<td>Methane</td>
<td>50</td>
</tr>
<tr>
<td>Carbon dioxide</td>
<td>0.95</td>
<td>Carbon dioxide</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Figure 10: component percentage with and without trapped air in canisters

CONCLUSIONS

The presence of mine air in the transport canister is an issue which may influence the gas content determination of the coal seam. This is particularly important when the transport canisters are not fully loaded with coal. The impact of the air column appears to influence released gas component percentages at both the Q2 and Q3 stages of Qm. Thus, it is important to address the issue of the trapped contaminant air when canisters are partially filled with coal cores. Possible ways of removing contaminant air include filling the void with inert material like Polymer rods, to be flushed with helium gas or flush the canister with the gas altogether. The alternative will be to use the % $O_2$ to determine the air-free gas percentage components. More significantly the air column length in the canister will have a decreasing effect on the gas mixture components percentage due to the much lower partial pressures of the coal gas in the canister during desorption.

ACKNOWLEDGEMENTS

This research work was undertaken with partial funding from Illawarra Coal. Advice and suggestions from Russell Thomas is acknowledged. Thanks to both Bob Seeley of West Mine and Jack Borg of Tahmoor Mine for providing freshly cored coal samples for the study. Special thanks to Murray Bull, Illawarra Coal - South 32, for his invaluable help in gas analysis at both the Q2 and Q3 stages.

REFERENCES


A CRITICAL REVIEW AND NEW APPROACH FOR DETERMINATION OF TRANSIENT GAS EMISSION BEHAVIOUR IN UNDERGROUND COAL MINES

Patrick Booth, Jan Nemcik and Ting Ren

ABSTRACT: Fugitive gas emissions from underground coal mining are forecast to increase beyond the practical management capacity of ventilation and current pre and post drainage systems. At the same time, investors, regulators and the community are demanding more accurate and evidence based emission information and transparent disclosure of carbon risk. The Paris Global Climate Change Agreement commits Australia to global emission reduction targets which aim to limit global warming to below 2°C with an aspirational goal of below 1.5°C (UNFCCC, 2015). This increases the scrutiny on coal operations, against the backdrop of lower prices, reduced margins and increased international competition. The prediction of methane emissions arising from underground coal mining has been the subject of extensive research for several decades, however calculation techniques remain empirically based and are hence limited to the origin of information in both application and resolution. Emission predictions are essential for the quantification and management of risk. To remain cost competitive and meet the Paris challenge, a step change to improvement is required in gas management. The identification and use of effective and timely controls for gas management will not only make mining safer, delivery of this outcome will reduce fugitive emissions, operational interruptions and thereby lift both coal and energy productivity.

A new approach to the determination of transient methane emission behaviour based upon fundamental physical and energy related principles is described. Operational risks and limitations associated with the present traditional approaches to gas emission prediction and design of gas management, such as localised and non-transferable empirical estimation of zone and degree of emission using historical data matching, assumptions that geological conditions will not change, and reduced spatial and time based resolution may be addressed using this fundamental basis.

INTRODUCTION

Australia possesses the fourth largest reserves of coal in the world. The production of these resources form a significant part of the Australian economy at an export value of over A$45B, providing direct employment for over 50,000 workers and over 3 per cent of Gross Domestic Product (Minerals Council of Australia 2015). Total black coal production is forecast to exceed 445 Mt in 2015-16, with the majority (98 per cent) of black coal production being from Queensland and New South Wales. Metallurgical coal exports increased by 4 per cent to 188 Mt during 2014-15, however 22 per cent lower unit prices achieved meant export value declined by 6 per cent to A$21.8B for the corresponding period. Similarly, thermal coal exports increased by 5 per cent to 195Mt and 15 per cent lower unit prices meant export value declined by 4 per cent to A$16.1B for the same period (Department of Industry 2015).

Australian coal mine fugitive emissions were 25 Mt CO₂-e in 2014–15; 66 per cent of total fugitive emissions of 38 Mt and 7 per cent of all Australian Greenhouse Gas (GHG) emissions of 560 Mt. From 2014–15 to 2019–20, coal mine fugitive emissions are projected to increase by 13 per cent to 29 Mt CO₂-e. (Department of the Environment 2015). Underground mines generally produce more fugitive emissions than surface mines because they contain more methane reflecting greater depths of cover. While significant growth in coal production is expected over the projection period, much of this is expected to be from surface mines with lower in-situ gas contents or through greater utilisation and efficiencies of current installed capacity.
Underground mining methods account for approximately 20 per cent of total Australian black coal production. Co-located with the coal reserves are significant quantities of methane gas (Geoscience Australia 2015) also known as Coal Seam Gas (CSG). Fugitive emissions of CSG via ventilation air not only contribute towards GHG inventory, but represent a lost opportunity for energy recovery as shown in Table 1. CSG reserves are not limited to economically recoverable coal seams, but also include coal measures above and below the working seams.

Table 1: Relative CO₂-e emissions and energy recovery of gas management alternatives

<table>
<thead>
<tr>
<th>Measure (Units)</th>
<th>Direct Vented</th>
<th>Vent Abated</th>
<th>Captured &amp; Flared</th>
<th>Captured &amp; Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pure methane (CH₄ litres/s)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Available Energy (GJ/hour)</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Net emissions (tCO₂-e/hr)</td>
<td>4.6</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>(0 if replacing grid sourced energy)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recovered Energy (MWh)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td>Benefit/s</td>
<td>NIL</td>
<td>Emissions reduced</td>
<td>Emissions reduced</td>
<td>Emissions reduced</td>
</tr>
</tbody>
</table>

The Paris Global Climate Change Agreement commits Australia to global emission reduction targets which aim to limit global warming to well below 2°C with an aspirational goal of below 1.5°C (UNFCCC 2015). To achieve this goal, investors, regulators and the broader community are demanding more accurate and evidence based emissions information. This increases the scrutiny on coal operations, against the backdrop of lower prices, reduced margins and increased international competition for the expected lower than previously forecast demand for coal as an energy fuel source.

Emission predictions are essential for the quantification and management of risk associated with; sudden gas release during mining (outbursts), accumulation of noxious or combustible gases within the mining environment, transport via drainage or ventilation systems, and utilisation or discharge to atmosphere. This paper focuses on prediction of transient methane emission due to longwall extraction for the purpose of managing accumulation of methane within the mining environment and increasing the potential for gas capture. The fundamental principles of the mechanisms of gas emission discussed in this review can be applied to prediction and management of risks of gas outburst. The broad potential application of the techniques described herein only increase the significance of this research.

The improved definition in the prediction of site specific transient gas emission character in terms of source location, quantity, composition, flow path and timing is acknowledged by several authors as critical for maintaining current production rates in higher gas content environments (Karacan et al., 2011; Packham et al., 2011 and Wang et al., 2011). Gas emissions will increase well beyond the practical management capacity of ventilation and current pre and post drainage systems at several Australian underground coal mines. Hence the traditional management approach of increasing ventilation quantity is unlikely to be sustainable due to practical constraints such as roadway area and maximum air velocity therein. Only a step change improvement in gas drainage, capture and utilisation practices will allow coal to remain a sustainable source of energy in a low emission world (Karacan et al., 2011). The identification and use of effective and timely controls for gas management will not only make mining safer, delivery of this outcome will reduce interruptions for reasons of safety management and lift both coal and overall energy productivity.

HISTORICAL GAS EMISSION PREDICTION

The prediction of methane emissions arising from underground coal mining has been the subject of extensive research for several decades (Curl 1978; Creedy 1993; Lunarzewski 1998 and Karacan 2008) and techniques range from simple geometric models to modern finite element models (Ashelford 2003).
and Guo et al., 2012). However, calculation techniques remain empirically based and are hence limited to the origin of information in both application and resolution.

Gas emissions due to mining extraction are transient and a complex function of:

- In-situ resource character
- The space where in-situ character and equilibrium is affected by extraction
- The degree to which character and equilibrium is affected, and
- System response

Specific Gas Emission (SGE) is a term commonly used in underground coal mining to describe the volume (in cubic metres) of pure gas expected to be released to the underground workings from all gas sources as a result of mining activity, expressed per unit tonne of coal mined. (Lunarzewski 1998) This should not be confused with absolute gassiness, which is an expression of the predicted or measured quantity of gas per unit of time.

The techniques for gas emission prediction have been standardised for the major coal producing regions around the world including Europe and the UK, America, Asia, Australia and more recently China (Creedy 1993; Lunarzewski 1998; Noack 1998 and Wang et al., 2012). Each of these regions has their own prediction variations; however most rely heavily on localised historical data for the underlying base measurement parameters and interpretation. Measurements required for the application of all techniques are essentially the same.

The most critical measurement in the application of all current gas emission prediction techniques is gas content measurement. Techniques for gas content measurement by both direct and indirect methods are described extensively in the literature and remain the subject of debate between researchers in relation to accuracy and repeatability of results (Saghafi et al., 2007). The high sensitivity of results to sampling technique and measurement conditions between in-situ and laboratory, combined with the limitations of laboratory processes for gas content measurements are described in the introductory section of AS3980-1999, “Guide to the determination of gas content of coal – Direct desorption method” (Australian Standards 2013).

The interpretation and calculation of firstly the zone of extraction influence, and secondly the degree, nature and timing of gas emission from that zone of influence has undergone successive improvement over the last seven decades and yet prediction methods remain empirically based. A pictorial summary of the historical improvement to prediction techniques is shown in Figure 1. Care must be taken when interpreting any of the diagrammatic versions of the predictive methods not to confuse the degree of emission with the spatial zone of emission.

![Figure 1: The Historical Gas Emission Prediction Journey](image-url)
The Flügge technique continues to be used for the purpose of total specific gas emission calculation at many Australian mines, however limitations in describing spatial and time based gas emission with any resolution renders it ineffective for design of gas drainage programs. Evidence provided through finite element analysis and micro seismic observations (Kelly et al., 1998) suggest the triangular prism representation shown in Figure 1 is only valid in specific geological conditions and does not cater well for changes in either geology or operational practices.

Improvements described by Noack (1998) and Curl (1978) included allowance for exponential reduction in degree of emission with increasing vertical distance into both the roof and floor. In the Winter model, recognition of the significant influence of strata strength in gas emission was first quantified with an empirical relationship. As shown in Figure 1, the zone of gas emission is represented as three separate curves with respect to either strong, medium or weak strata. A consolidated review of the state of gas emission prediction research at the time was presented in Curl (1978). The review compared up to 10 different predictive techniques and argued that in most cases the techniques were only suitable for the regions in which they were developed due to the reliance on empirical factors. Curl argued that of all the predictive models researched, only the UK’s MRDE technique, based on Airey (1968), presented a coherent explanation of the influences to gas emission. Corrections for depth, extraction rate, front abutment distance, and weekly production cycle were all included in the calculation.

Rather than making minor changes to existing zone of emission and degree of emission curves, research by various workers (Gray 1987; Barker-Read and Radchenko 1989 and Lama and Bodziony 1998) focussed on understanding of fundamental mechanisms driving gas emission behaviour from coal and surrounding strata. The importance of cleat and joint geometry and net effective stress in the control of fluid movement was highlighted. Research by Lama in particular led to the significant reduction of risk associated with gas outburst through the development of gas content threshold levels for the Bulli seam. The thresholds largely remain in place in the Australian coal industry to the present day due to the fundamental nature and sound principles based methodology used in that research. These include methods based upon differential sorption properties of coal under the effect of a shear structure, and gas pressure measurements which change as a result of changes in the permeability of the structure. Significantly, the fracture density and sorption properties may change up to 20 m away from the shear structure, but gas pressure changes can occur up to 100 m away from the structure.

The GeoGAS Longwall “Pore Pressure” model described by Ashelford (2003) took account of many gas reservoir and geological parameters of coal seams and allowed variation of mining operations in arriving at a gas emission value. The calculation process involves changes in “pore pressure” resulting from rock mechanics as shown in Figure 1. The model relies upon measured gas reservoir properties for the determination of gas release such as; measured gas content ($Q_m$) at reservoir temperature, gas desorption rate, gas composition, gas sorption capacity at reservoir temperature, seam thickness and mineral matter above and below the working section, pore pressure and coal and sandstone porosity, and therefore the model outputs are largely based in fundamental scientific principles. The parameters and how they are measured are described by Williams et al., (2001).

Definition of the pore pressure resulting from mining was obtained as an output from the finite element analysis software FLAC (ITASCA 1995). The advantage of this model over other was its ability to accurately predict the magnitude of gas emission from the floor seams below the Bulli seam in the southern Sydney basin at mines such as West Cliff and Appin. This was due to the significant deformation and order of magnitude changes in horizontal and vertical stress in the floor strata recognised and displayed by the FLAC software. Whilst the pore pressure model remains the most adaptive and fundamentally based calculation of gas emission for longwall operations, the assumptions limit the application of this technique to the increasing spatial and time resolution required for design of gas drainage programs. Limitations include; sudden gas release from floor breaks cannot be modelled, desorption behaviour being consistent with the gas desorption isotherm for that particular coal type, and the assumption that gas desorption is the limiting factor in the supply of gas to the system.
The availability of increasing computational processing capability enabled the management of the increased size and complexity of the data available for gas emission analysis through the later part of the last decade. Studies by (Karacan, 2008; Karacan and Goodman 2012 and Karacan and Olea 2014) used statistical, Principle Component Analysis (PCA) and Artificial Neural Network (ANN) based approaches to predict the ventilation methane emission rates of U.S. longwall mines. Critically, all techniques which involve the use of large historical data sets for gas emission prediction by analysis by statistical, PCA or ANN approaches rely on a fundamental assumption that input conditions will not materially change.

Gas emission prediction in China is generally derived by a statistical method, where localised historical data is available, or a split source sum of the contributing parts method when specific data is unavailable. Chinese standards (AQ1018-2006) allow either approach to be used by trained specialist engineers familiar with the local conditions for which the prediction is made. The statistical approach makes several assumptions regarding the consistency of geology and operating practices in the locality of prediction. The use of linear interpolation of gas environmental input data with increasing depth of mining in the split source method and its limitations are discussed by (Chen et al., 2014).

In summary, all historical gas emission prediction techniques follow variations of the same basic steps to arrive at a broad ranging result in terms of SGE in cubic metres of gas per tonne of coal mined. Spatial and time resolution of any prediction technique following this basic methodology is limited, and the result only applicable to the particular conditions in which the prediction is made. Comparison of various prediction models is difficult for the reasons discussed in Jensen et al (1992).

**RELEVANT GAS GENERATION, STORAGE AND FLOW FUNDAMENTALS**

Coalbed gas or coal seam gas are general terms used to describe gases contained within coal measures that are generated as part of coalification and other geological and hydrogeological processes. Similar to the creation of coal itself, coal bed gas generation pathways are also dependent on fundamental physical and chemical character and changes in both level and form of energy within the environment. Coal bed methane can be classified as either biogenic or thermogenic in origin and may be differentiated by carbon isotopic composition amongst other techniques. (Burra et al., 2014) summarised the key isotopic interpretations, however (Pashin 2014) cautions against the strict use of carbon isotope value thresholds for the determination of methane origin because biogenic gas generated by CO₂ reduction may produce larger δ¹³C values depending on the source of the original CO₂ consumed by the methanogens. Saghafi et al., (2015) recently provided field results for a mine in the southern Sydney basin which, when combined with traditional chemical compositional analysis for ratios of methane and carbon dioxide to higher hydrocarbons, suggests that isotopic analysis is a valid technique to differentiate between gas sources in multi seam environments.

Biogenic methane is generated at low temperature by anaerobic microbes (methanogens) when coal beds are exposed to groundwater recharge after basin deformation. The dominant biological processes involved in the generation of biogenic methane include carbon dioxide reduction and acetate reduction or fermentation which are described in chemical equations (1) and (2a) and (b). Two significant factors must be carefully considered in the characterisation of the origin of biogenic gas. Firstly, for carbon dioxide reduction to methane, Hydrogen must be present. Secondly, in addition to the methane, the two part acetate fermentation process also produces CO₂.

\[
\begin{align*}
\text{CO}_2 + 4H_2 & \rightarrow CH_4 + 2H_2O \\
\text{CH}_3\text{COO}^- + H^+ & \rightarrow CH_4 + HCO_3^- \quad \text{(a)}
\end{align*}
\]

Availability of hydrogen ions is increased via groundwater flow and recharge in subterranean aquifers. The flow pathway of water is therefore an important factor in characterising gas reservoir conditions. The relative rate of change of coal seam gradient provides information on available potential energy under the influence of gravity. The effect of gravity on hydrogeological character has remained constant over geological time.
Thermogenic gas is generated at high temperature during late stage coalification and generally contain heavier carbon isotopes than biogenic gas. The results described from (Moore 2012) indicate that the first gas generated via thermogenic processes is CO\textsubscript{2} at approximately 50 °C. Above this temperature, increasing amounts of hydrocarbons (methane, ethane and higher) and nitrogen are produced at maximum volume at approximately 150°C. At higher temperature, gas generation reduces, producing a parabolic maximum gas volume trend with temperature and/or rank. Such parabolic gas content trends have been reported from a number of Australian Basins (Faiz et al., 2007).

Over ninety per cent of gas storage in coal occurs by physical adsorption to the surface of the coal matrix, including the surfaces of all internal pores and cleats or fractures. (Flores 1998) The remaining is free gas, which may also reside within internal pores depending on pore geometry, and also within cleats or fractures. It is the physical adsorption process which differentiates coal bed reservoirs from conventional gas reservoirs. Conventional gas reservoirs may contain only one-sixth to one-seventh of the equivalent coal bed reservoir by rock volume, as the gas is free within porous spaces and not held to surfaces via adsorption.

The fundamental adsorption concepts between gas and a solid surface are usually described in terms of isotherms, where the amount of adsorbate on adsorbent is shown as a function of pressure at constant temperature as depicted in Figure 2. Up to 15 models for the adsorption principles are described in the literature, the most common being the Langmuir model. Langmuir suggested that adsorption takes place through the equilibrium mechanism. Thus at low pressures this molecularly denser state allows greater volumes to be stored by sorption than is possible by compression, however the availability of adsorption sites (surface area) remains the key parameter. Higher pressures increase the potential that a particular molecule will find an adsorption site. Other extended formulations of the Langmuir model to address some of the more practical limitations, such as presence of mixed gases, mineralisation and moisture are described in the literature (Moore 2012).

![Figure 2: Typical gas isotherm](image)

The movement of gas molecules through other gases, fluids or solids are described by Fick’s Laws. Both of these laws are of relevance to the prediction of gas emission from coal and are demonstrated in Table 2. Fick’s first law relates the diffusive flux to concentration under the assumption of steady state. Fick’s second law predicts how diffusion causes the concentration to change with time. It is a partial differential equation where concentration per unit volume is a function that depends on location and time. The key point for diffusive behaviour is that the energy driving the diffusion process is atomic energy and molecular vibration motion in response to this energy. The concentration gradient is a proxy term for molecular energy density per unit volume gradient across three dimensional space, and is relatively small in energy terms in the absence of other forces (e.g. pressure gradients). Solutions for
either time or distance can be obtained from evaluation of both of Fick’s Laws simultaneously. The critical point being that on reducing spatial component of both equations (dx), it becomes more probable that molecules will be subject to much larger external energy forces (e.g. pressure gradients) in shorter timeframes.

Darcy’s law is an expression of conservation of momentum and was initially determined experimentally. At constant elevation, Darcy’s law is a simple proportional relationship between the instantaneous discharge rate through a porous medium Q, the viscosity of the fluid u and the pressure drop (P_a – P_b) over a given distance L as shown in Table 2. This equation can also be solved for permeability, allowing for relative permeability to be calculated by forcing a fluid of known viscosity through a core of a known length and area, and measuring the pressure drop across the length of the core. In practice, this measurement is difficult and expensive to complete in-situ, but is the only method of obtaining a true permeability result which reflects the reservoir conditions.

In case of coal, permeability is a complex, multi-dimensional function of several influences such as width, length, height, aperture spacing, frequency or density, and connectivity of cleats or fractures (Flores 2014). Many of these influence functions are non-linear, however, similar to the use of the “impedance” in electrical circuit theory, have components that can be either readily measured directly or indirectly or otherwise grouped without affecting materially affecting calculation results. Changes in permeability in coal may be summarised into two main components; firstly the effective stress effects, and secondly the shrink and swell strain effects on the coal matrix with desorption or adsorption which may increase or decrease relative permeability, both shown mathematically in Table 2.

Coal composition controls a broad range of gas reservoir properties including gas adsorption capacity, gas content, porosity, permeability and gas transport. The gas storage capacity in particular is defined by coal structure. Using fundamental physical, chemical, energy and geometric relationships, it is postulated that for the purpose of gas emission prediction, dynamic response of the gas reservoir to mining extraction can be reliably predicted using measured data which is largely available through proximate characterisation parameters such as rank, carbon content, macerals, and moisture content.

Table 2: Common form descriptions for various forces involved in gas emission

<table>
<thead>
<tr>
<th>Term</th>
<th>Adsortion</th>
<th>Diffusion</th>
<th>Bulk Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applicable Law</td>
<td>van der Waals</td>
<td>Fick’s</td>
<td>Darcy’s</td>
</tr>
<tr>
<td>Formulae</td>
<td>$F_{W}(r) = \frac{4Rc}{(Rc + R_h)^2}$</td>
<td>$J = -D \frac{\partial \phi}{\partial x}$</td>
<td>$Q = \frac{-k \cdot \Delta (P_a - P_b)}{\mu L}$</td>
</tr>
<tr>
<td>Traditional tools</td>
<td>Langmuir Isotherm</td>
<td>$D = D_o \exp \left( - \frac{Q}{RT} \right)$</td>
<td>Permeability determination</td>
</tr>
<tr>
<td>Key influences</td>
<td>Surface area (no. of sites) &amp; size of atoms involved</td>
<td>Concentration gradient over distance (dx) and substances</td>
<td>Effective flow path length and size (cubic relation)</td>
</tr>
<tr>
<td>Key measurements</td>
<td>Volume versus Pressure at constant temperature</td>
<td>Diffusion time and amount by experiment</td>
<td>Packer / long term draw test</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Lab based experiment</td>
</tr>
</tbody>
</table>

The coal structure sets adsorption/desorption character, which also changes with gas type, but this does not necessarily mean that a coal of certain properties and whose sorption capacity is described by a particular isotherm actually contains that amount of gas for a given volume. The ratio between actual gas content and the theoretical capacity is known as the degree of saturation and is expressed as a percentage. Lower in-situ degree of saturation is an indicator that other mechanisms, such as lowering of hydrostatic pressure through fluid movement, have potentially allowed gas to migrate or otherwise be released from the coal after initial gas generation. Mineralisation influences internal structure, geometry,
pore availability to gas adsorption, ability of gas to flow, shrinking and swelling, gas content, gas recoverability, and potential for enhanced gas recovery (Flores 2014).

A common technique for characterisation of the coal structure is pore size distribution analysis. Pore sizes are categorised into micro, meso and macropores with sizes less than 2nm, 2-50nm and above 50nm respectively. Experimental evidence closely correlates increasing coal rank with higher proportions of micropores. (Mosher et al., 2013). An increase in micropore distribution per unit of coal volume also infers an increase in surface area available for gas adsorption sites, in turn explaining observed experimental increase in gas storage capacity with coal rank, and increase in rate of change of volumetric capacity per unit pressure change as described by Kim in the review by (Moore 2012).

A NEW APPROACH

The fundamental nature of these physical and chemical interactions between the principal components of coal, coal seam gas and other substances found in the mining environment have remained constant over geological time and are significantly influenced by the various forms of energy applied. Only with recent improvement to analytical and measurement tools and processes, in some cases down to the sub-atomic scale, can the interactions which are applicable to the mining environment be appreciated. Some of the basic atomic properties relevant to the mining environment are demonstrated in Table 3, specifically noting the relative size and electron affinity of Carbon compared to other atoms.

<table>
<thead>
<tr>
<th>Element name</th>
<th>Atomic number</th>
<th>Shell configuration</th>
<th>Valence electrons</th>
<th>Radius (pm)</th>
<th>Electro-negativity</th>
<th>Electron-affinity (kJ/mol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrogen</td>
<td>1</td>
<td>1s</td>
<td>1</td>
<td>1</td>
<td>-2.20</td>
<td>72.8</td>
</tr>
<tr>
<td>Helium</td>
<td>2</td>
<td>1s</td>
<td>0</td>
<td>2</td>
<td>-0.80</td>
<td>0</td>
</tr>
<tr>
<td>Carbon</td>
<td>6</td>
<td>1s 2s 2p</td>
<td>4</td>
<td>67</td>
<td>2.55</td>
<td>153.9</td>
</tr>
<tr>
<td>Nitrogen</td>
<td>7</td>
<td>1s 2s 2p</td>
<td>5</td>
<td>56</td>
<td>3.04</td>
<td>7</td>
</tr>
<tr>
<td>Oxygen</td>
<td>8</td>
<td>1s 2s 2p</td>
<td>2</td>
<td>48</td>
<td>3.44</td>
<td>141</td>
</tr>
</tbody>
</table>

An indication to the application of energy based physics in the determination of transient gas emission behaviour is provided in the relationships summarised in Table 2. These relationships can be used to describe the complex interactions between atomic particles at sub Pico metre ($10^{-12}$) scale and may also be applied to coal gas interactions at the molecular scale. This is due to well defined and understood geometric and energy properties of the typical gases, liquids and solids involved. Recent literature and experimental results support the use of an energy based approach (Mosher et al., 2013).

Van der Waal’s forces are the sum of the attractive or repulsive forces between molecules. The term includes up to four different types of generally anisotropic forces which depend on the relative orientation of the molecules. Table 2 shows the van der Waals force $F_{vw}(r)$ between two spheres of constant radii $R_1$ and $R_2$ is a function of the size of the spheres and the separation distance, $r$. The force on an object being the negative derivative of the potential energy function. Evaluation of the several potential molecular interactions between coal matrix, gases, and surface and pore geometries suggests a plausible explanation for preferential adsorption and increased sorption capacity of CO$_2$ over CH$_4$, and reduction of adsorption capacity with increase in moisture content for any coal type by reference to gas atomic and molecular parameters shown in Tables 3 and 4.

Fick’s Laws commonly describe diffusive processes between gases, gases and liquids, and are generally characterised by reference to a concentration gradient. However, in the absence of pressure difference and bulk molecular flow, the only source of energy for diffusion is provided by molecular vibration. Observation of molecular geometry combined with basic reservoir characterising parameters (pressure, temperature and phase state) will thus provide data to inform the likely behaviour. Darcy’s Law, complete with various extensions, is used to describe bulk molecular flow under the driving energy of pressure differential and may be applied to single phase gas or two phase gas-liquid flow. The timing of pressure differential is significant in the application to understanding transient coal gas
emission behaviour, as the combination of time and energies available will determine the changes to various equilibriums throughout the processes.

**Table 4: Common molecular properties**

<table>
<thead>
<tr>
<th>Common molecule name</th>
<th>Chemical Symbol</th>
<th>State at STP</th>
<th>Lewis Structure</th>
<th>Space fill model</th>
<th>Molecule size (nm)</th>
<th>Density (kg/m³)</th>
<th>Molar Mass (g/mol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Methane</td>
<td>CH₄</td>
<td>Gas</td>
<td><img src="image" alt="Lewis Structure" /></td>
<td><img src="image" alt="Space fill model" /></td>
<td>0.38</td>
<td>0.72</td>
<td>16.0</td>
</tr>
<tr>
<td>Nitrogen</td>
<td>N₂</td>
<td>Gas</td>
<td><img src="image" alt="Lewis Structure" /></td>
<td><img src="image" alt="Space fill model" /></td>
<td>0.36</td>
<td>1.25</td>
<td>28.0</td>
</tr>
<tr>
<td>Carbon Dioxide</td>
<td>CO₂</td>
<td>Gas</td>
<td><img src="image" alt="Lewis Structure" /></td>
<td><img src="image" alt="Space fill model" /></td>
<td>0.34</td>
<td>1.98</td>
<td>44.0</td>
</tr>
<tr>
<td>Water</td>
<td>H₂O</td>
<td>Liquid</td>
<td><img src="image" alt="Lewis Structure" /></td>
<td><img src="image" alt="Space fill model" /></td>
<td>0.27</td>
<td>998</td>
<td>18.0</td>
</tr>
</tbody>
</table>

Strata response to extraction is defined by strata and coal properties and geometry of the extraction. These processes may be modelled using commercially available finite element software. The application of outputs of such software may be used to inform gas emission characterisation, particularly with respect to stress history and magnitude, and fracture extent and nature. Strata failure modes of relevance to longwall extraction and consequent gas emission have been predicted and verified under a range of geological conditions using various integrated techniques (Kelly et al., 1998). In particular, the orientation and properties of fractures including; shear fracture through intact rock and along bedding, and tensile fracture of intact rock and along bedding planes were accurately forecast and measured in spatial and time dimensions. The depth and nature of stress change and fracturing into the floor below, particularly for the case study results given, suggests that previous methods for gas emission prediction may significantly underestimate the contribution from underlying seams where they exist.

Excepting longwall block commencement and completion zones, and in the absence of other geological structures, the effect of stress history and fracturing may be repeatable and not require continuous recalculation. A computationally efficient method of testing for recalculation requirements using accessible data has been trialled. Key indicators for recalculation requirements are associated with; rate of change of gradient (i.e. \( \frac{d^2z}{dx, dy} \)), rate of change of stress and stress history, change in fracture mechanism (tension, compression, and shear), and rate of change of pressure (i.e. \( \frac{dp}{dt} \) and \( \frac{dp}{dx} \)).

**RECENT SUPPORTING LITERATURE AND EXPERIMENTAL RESULTS**

Molecular modelling provided detailed simulation of physical adsorption processes between gas and coal (Mosher et al., 2013). This work demonstrates that relative geometry and size of molecule to pore is critical. The results demonstrated through simulation and displayed in Figures 3a and 3b clearly support the influence of physics and atomic properties such as those shown in Tables 3 and 4. When combined with pore size distribution data, the results explain many laboratory and field observations, such as those found in Flores (2014).

The application of Finite Element Analysis (FEA) software and CFD coupling to gas emission prediction (Guo et al., 2012) demonstrates that fundamental principles can be used for the prediction of gas emission. Magnitude of stress change is the key parameter for determining permeability change. While the field trials using continuous coupling between FEA and CFD models performed in the “Cosflow” software have shown good correlation to calculation results, the process is computationally exhaustive.

Recent use of multiple experimental techniques to determine permeability evolution under cyclic load (Cai et al., 2014) shown in Figure 5, clearly demonstrates the roles energy change and history have in determining transient emission behaviour. This study used X-Ray CT and acoustic emission results from a coal sample subjected to cyclic loading in a tri-axial test apparatus and demonstrated that on initial
increase in effective stress, permeability was reduced. However, during this increase in load, X-Ray CT results also demonstrated the initial creation of micro-fracturing within the sample. Successive increase in load on the sample eventually increased the density of fracturing to a point of failure where permeability increased exponentially. It is the relationship of initial increase in stress to creation of micro-fracturing within coal that is to be further examined by experiment, along with the evolution of fracture aperture and eventual role in determining pressure change in the underground environment.

**Figures 3(a) and (b):** Adsorption simulation after Mosher et al., (2013)

**Figure 4:** Examples of the application of FEA modelling techniques after Guo (2012)

**Figure 5:** Determination of permeability evolution under cyclic load after Cai et al., (2014)
CONCLUSIONS

Accurate characterisation of transient gas emissions in terms of quantity, composition, source location and timing – particularly during longwall extraction - is essential to reduce uncertainty in the site specific evaluation and optimisation of upstream mine gas management practices and hence improve safety and sustainability, enhance production and reduce cost.

The character of the initial gas reservoir and its likely response to longwall extraction is largely determined by the structure of the gas bearing stratum (may be coal, shales or porous sandstones) and the types and magnitude of energy that stratum has been subject to over geological time. Conversely, the rate, quantity and timing of gas emission during longwall extraction is determined by energy transfer and exchange as a result of operational practices occurring over a period ranging from seconds to years. Gas adsorption, diffusion and flow (source) character within a potential gas emission space has been traditionally informed through the use of gas content and composition testing. The continued use this testing as the sole means of quantifying inputs to emission prediction calculation will remain a key limiting factor in obtaining the spatial and time based resolution required for optimal design of gas drainage practices.

All prediction techniques which involve the use of large historical data sets for gas emission prediction by analysis by statistical approaches rely on a fundamental assumption that input conditions will not materially change – particularly with respect to geology, gas content and composition within that geology, and extraction practices. A new approach to emission prediction is proposed, incorporating fundamental energy related principles and calculation techniques associated with; rate of change of strata gradient, rate of change of stress and stress history, and change in fracture mechanism and rate of change of pressure with respect to both space and time. For application to initial ventilation system design, broad range emission prediction may suffice. However, where ventilation capacity is constrained and gas drainage design is required for mitigation of risk, a dynamic and high resolution technique is required. This will enable step change improvement in gas management practices to be delivered sustainably.

The reduction of total net gas management life cycle costs, firstly in operation of high volume ventilation arrangements with conservative capacity design bases, and secondly in the execution of high density drilling in gas drainage programs which frequently do not deliver increased gas capture rates or result in drainage of gas below relevant thresholds in the timeframes required, is a key required outcome.

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REFERENCES


AN INNOVATIVE DRAINAGE SYSTEM FOR COAL MINE METHANE CAPTURE OPTIMISATION AND ABATEMENT MAXIMISATION

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ABSTRACT: Gas drainage in Australian longwall mining is increasingly challenging because of complex gassy conditions, multi-seam environments, and environments where drilling a large number of conventional surface vertical gas drainage boreholes is not practical. This paper presents an alternative approach, using horizontal boreholes for longwall goaf gas drainage. This horizontal drainage method has been trialled successfully at the Blakefield South mine. The trial results show that horizontal drainage boreholes have significantly improved longwall gas management, through controlling goaf gas pressure distribution and flow pattern. As a result, gas related longwall production delays were largely eliminated and the fugitive emissions from the mine were significantly reduced.

INTRODUCTION

Gas drainage in longwall mining in Australia is becoming increasingly challenging and complex with deeper mines, multi-seam environments beneath existing goafs, and surface environments where drilling conventional surface gas drainage wells is more constrained. Increased longwall retreat and development rates at a number of Australian coal regions, including the Hunter Valley, Illawarra and Bowen Basin have produced mine gas levels that are a serious threat to sustained and efficient coal production, and will potentially lead to increased fugitive emissions.

A step-change improvement in coal mine methane drainage strategies and technologies is required to effectively capture methane from the longwall goaf, surrounding seams and strata, before the methane enters the workings and ventilation system. Such an improvement will also enhance mining safety and remove one of the most significant barriers that constrain mining efficiency at Australian gassy mines now and in the future, particularly as mining depth increases.

A collaborative research project between the Glencore Bulga Underground Operations (hereafter referred to as "Bulga") and the Commonwealth Scientific and Industrial Research Organisation (CSIRO) has been carried out, under the Australian Government Coal Mining Abatement Technology Support Package (CMATSP). The project aims to develop and demonstrate a holistic and optimal approach to planning, design and operational control of mine gas drainage to maximise methane capture and minimise fugitive emissions in gassy and multiple seam conditions. The project commenced in August 2013, and to date the major part of the work has been completed.

This paper presents the methodology and approach used and the innovative gas drainage system that resulted from the project.

DESIGN METHODOLOGIES AND PROCEDURES

A key objective of gas drainage design is to provide sufficient drainage capacity at optimally positioned drainage points that not only captures an optimal fraction of goaf gas emissions at an acceptable composition but also reduces methane emissions into ventilation (Guo et al., 2012 and 2014). To date,
the development and optimisation of a mine gas drainage design has been done largely by a trial-and-error method and has mostly proven to be inefficient and costly.

Gas emission during longwall mining is a dynamic process which involves interactions between mining induced coal and strata fracturing, de-stressing, de-watering, ventilation and gas drainage. Therefore, a clear understanding of the coupled strata, gas and underground water behaviours during longwall mining is crucial to determine key parameters from gas drainage design, such as gas emission source and emission patterns. An integrated approach consisting of site characterisation, field studies and numerical modelling was developed and carried out. The approach consists of four key aspects:

- site characterisation;
- integrated field studies;
- geomechanical and coupled numerical modelling; and
- Computational Fluid Dynamics (CFD) simulations.

Site conditions

The Blakefield South longwalls operate under old mine workings in highly gasy conditions and a multiple of seam environments. The working seam, Blakefield, dips at about 3 degrees and its depth of cover varies between 130 and 260 m. The seam comprises a number of plies with a total thickness ranging from 4.5 to 8.0 m. The extraction height ranges from 2.8 to 3.4 m. The LW3 panel, selected for field and numerical studies is 400 m wide (rib-to-rib) and about 3.5 km long. Figure 1 shows the mine layout, where LW3 is to be mined, and Figure 2 shows a stratigraphic section about LW3.

![Mine plan of Blakefield South mine. The panel with boreholes shown is the LW3 panel](image)

Some key observations from the site characterisation include:

- The overburden is highly banded, consisting of sandstone, conglomerate, siltstone, shale and tuff. No thick and competent layers such as sandstone overlie or underlie the mining seam within the depth of interest.
• No major geological structures such as faults and dykes that have a significant influence on gas reservoir conditions are present.
• The old Whybrow goafs, overlying around 70 m above the mining level, significantly influence goaf gas drainage performance. It is evident that a significant portion of the goaf gas drained by surface vertical wells was from the Whybrow goafs.
• The performance of surface vertical wells is not satisfactory in controlling goaf gas emissions, particularly in the initial 400 m from the longwall start-up. Frequent longwall production delays were experienced during the first 400 m retreat of LW2 and LW3 which totalled about two months.
• Some surface vertical wells to LW2 were operated for only a few days due to unacceptable high levels of oxygen in the drainage gas.

![Stratigraphic section of the Blakefield South Mine](image)

**Figure 2: Stratigraphic section of the Blakefield South Mine**

**Integrated field studies of coupled strata, gas and water**

The objective of these comprehensive and integrated field studies is to understand the coupled behaviour of strata, gas and groundwater during longwall mining. The studies cover overburden caving processes and strata movement, surface subsidence, surface vertical well stability, extent of mining influence, goaf gas pressure dynamics changes, gas flow patterns, gas content change before and after mining, and gas drainage performance under various operational parameters.

Key findings from the studies include:

• Mining induced fractures and delaminations extend up to the Redbank Creek seam quickly (in 36 m inbye the longwall face). Significant delaminations were observed in the interburden. This process indicates that gas would release rapidly from both overlying coal seams. Figure 3 shows the monitored overburden movement by one extensometer as an example.
• A piezometer, installed at a location 85 m away from the mining block, shows coal seam pore pressure decreases quickly between 50 m outbye and 100 m inbye of the longwall face. In the vertical direction, coal seams other than Woodlands Hill incur a significant decrease of pore fluid pressure.
- Surface goaf gas drainage borehole inspections with a borehole camera clearly show that the goaf drainage boreholes were often blocked at levels higher than 30 m above the mining seam.
- The results indicate that longwall gas emission sources are the Redbank Creek and Wambo seams in the roof and the Glen Munro seam in the floor, which is different to that predicted by Flugge Model with Woodlands Hill seam being a significant emission source.

![Figure 3: Monitored overburden movement above LW3](image)

**Geomechanical and coupled modelling studies**

The objective of coupled numerical modelling is to extrapolate the field studies results into a 3D dimensional space and provide key parameters for the consequent CFD simulations and the optimisation of goaf gas drainage design. The shape and extent of the caved and fractured zones and the permeability changes and distributions within these zones are assessed from the geomechanical modelling, which is validated from field monitoring results. The coupled modelling is then further carried out to determine the shapes of gas emission zones and various gas emission sources.

The numerical studies were carried out with a CSIRO developed computer code COSFLOW. Further information about this code can be found in reference (Guo, et al., 2009). Figure 4 shows the 3D geometry model constructed for the numerical modelling. Figure 5 shows the predicted de-stressing zone and the permeability changes. Figure 5 also shows a result of the gas emission region at Wambo seam.

![Figure 4: 3D geometry model used for COSFLOW modelling studies](image)
CFD simulations of goaf gas drainage mechanisms and design optimisation

CFD modelling has been an important tool at CSIRO for studying goaf gas flows and optimising goaf gas drainage (Balusu, 2002). In this research, the CFD modelling is used to understand goaf gas flow patterns, gas drainage mechanisms with different means such as horizontal boreholes and vertical boreholes, and parametric studies for gas drainage optimisation. The results have clearly revealed drainage mechanisms under various means and provided valuable information to assist the optimisation of gas drainage design.

Figure 6 shows the CFD model constructed for the simulation of LW3. The parameters used in the model were based on field data including the ventilation rate, pressure difference across the longwall face, drainage parameters and gas emission rate. The model was calibrated by the methane concentration and flow rate in various surface goaf vertical wells as well as by current knowledge of goaf gas pressure distribution.

Taking the horizontal borehole drainage as an example, Figure 7 (a) shows the pressure contour on a vertical section 20 m behind the face. It is seen that sinks of low pressure are formed around the boreholes. Figure 7 (b) further shows that, under the low pressure sinks, gas is induced to flow towards the drainage boreholes away from the workings. These results illustrate that the horizontal drainage boreholes create low pressure sinks that protect the workings from goaf gas ingresses by changing goaf gas flow directions. Given the horizontal boreholes traverse along the direction of the longwall advance, the boreholes would provide continuous and consistent drainage mechanisms and capacity as the longwall advances. This system differs from using vertical boreholes which usually have varied drainage flow rate during mining.
DESIGN AND TRIAL OF A NEW GOAF GAS DRAINAGE SYSTEM

Design

The field and numerical studies provided a clear understanding of site conditions, technical issues with the mine’s surface vertical goaf wells, coupled behaviour of strata, gas and water, and gas emission sources. As a result, an innovative horizontal drainage was proposed and designed. The design is constructed on the basis of the following key principles and considerations:

- provide continuous and immediate capture of gas emissions before they reach the ventilation circuit;
- locate in the return side of the longwall to capture rich gas and maximise gas drainage efficiency;
- locate slightly above the caved zone to maintain borehole connection to the goaf and stability; and
- avoid drilling into any soft strata layers such as clay and coal seams.

A design of underground lateral boreholes was then made as shown in Figure 8. In the design, the lateral boreholes are drilled from underground to reduce the risk of borehole blockage and collapse. Five boreholes in the roof and five boreholes in the floor were included. The roof boreholes were located...
within 150 m of the ventilation return and vertically situated at a location between 13 to 20 m above the mining seam. The floor boreholes were designed to steer along the Glen Munro seam, with three boreholes located in the ventilation return side and two boreholes in the intake side. The floor boreholes aimed to reduce the Glen Munro seam gas flowing into the goaf.

Figure 8: Design of horizontal gas drainage system with underground lateral boreholes.
Drainage design implementation and monitoring management

The design was trialled at LW4 at its initial 400 m of retreat. The implemented borehole layout is shown in Figure 9 (a). The cross sections of the lateral boreholes are shown in Figure 9 (b) and (c), respectively. The two groups of roof and floor boreholes were each connected to a riser, drilled from the surface to a cut-through point. Each riser was 305 mm in diameter and connected to the goaf drainage plant situated on the ground surface to provide suction pressure to the underground lateral boreholes.

The configuration of these boreholes is summarised below:

- Five roof lateral boreholes were located within 105 m from the ventilation return roadway, with the nearest one 25 m from the longwall void edge. The spacing between every two boreholes was about 20 m.
- The roof boreholes were situated at 15-22 m above the mining seam, with the first one from the return the lowest.
- The horizontal section covered a distance of about 350 m.
- The roof boreholes were 145 mm in diameter, reamed from 96 mm.
- Five floor boreholes were drilled in the Glen Munro seam. Three were located close to the return side (26 m outside the LW4 panel to 105 m inside the LW4 panel), and two in the intake side (75 m of the maingate).
- The floor lateral boreholes were not reamed and were 96 mm in diameter.

Continuous monitoring of gas drainage performance was carried out. Both risers were equipped with a wellhead, which enabled continuous monitoring of suction pressure, drainage gas flow rate, gas composition, and operation parameters such as borehole opening percentage. Tube bundles were run down and connected to each of the roof lateral boreholes to monitor gas composition. Continuous monitoring of individual borehole flowrate was not enabled but manual readings were taken. In addition, a test of gas drainage performance with different operational parameters, such as various boreholes in operation and borehole opening percentage, were conducted.

Surface vertical wells were implemented in the remaining part of the LW4. There were also three surface vertical wells drilled within the trial area for a transition from horizontal drainage to vertical drainage. The first one, 4C, was located 240 m from the longwall start-up.

TRIAL RESULTS

Gas drainage performance

Roof lateral boreholes

Figure 10 (a) and (b) show the drainage flow rate and methane concentration in the first two months of LW4 operation, which covers both the underground lateral boreholes and the conventional surface vertical wells and compares the performance between the two means. It can be seen that:

- The roof lateral boreholes captured goaf gas with a continuous and consistent flow rate; while the vertical goaf wells gas flow rate fluctuated significantly.
- Methane concentration in the roof lateral boreholes was high and averaged 86%; while in the vertical wells, methane concentration averaged 68.3% and varied significantly.
- Methane concentration in the ventilation return was remarkably lower when the roof lateral boreholes were solely in operation (average 1.13 %) than that when goaf vertical wells were solely in operation (average 1.61%). This clearly shows that the roof lateral boreholes significantly reduced methane emissions from longwall operation into the ventilation circuit.
- Overall, in comparison to the goaf vertical wells, the roof lateral boreholes captured less gas but achieved a better result in controlling gas emissions into the ventilation circuit. When compared
to the surface vertical goaf wells, the roof lateral boreholes enabled gas to be captured from an area where it is more critical to goaf gas control.

(a) Layout of the trialled innovative goaf gas drainage system at LW4 (b) cross-section along mining face

(c) Cross-section along mining direction

Figure 9: Implemented innovative gas drainage system at LW4 as a trial
Figure 10: Gas drainage performance of the innovative gas drainage system at the LW4 trial. (a) drainage flow rate; (b) drainage methane purity

**Floor laterals boreholes**

It is noted that the floor lateral boreholes in the Glen Munro seam were not connected to the goaf suction plant and the gas flow was driven by seam gas pressure only. No monitoring of flow rate and methane concentration was implemented for individual boreholes but the combined flow rate and gas composition was continuously monitored.

The floor lateral boreholes performed well with continuous and stable gas flow and consistently high methane purity (92%, the rest being mainly CO₂) before and during mining. The gas flow rate from the lateral boreholes was at about 400-450 l/s. A tracer gas test, where SF6 was injected into one of the floor boreholes, showed that no SF6 was captured in the ventilation circuit and drainage boreholes. This indicates the floor boreholes have effectively prevented Glen Munro seam gas from flowing up into the goaf and ventilation return.
Improvement of drainage efficiency and longwall coal production

Gas drainage efficiency has been used as a key factor to assess gas drainage performance and its effectiveness in controlling goaf gas emissions into the ventilation circuit. Gas drainage efficiency is calculated as a percentage of the drainage gas volume in the total gas emissions from mining operation. The gas drainage efficiency during the trial period was significantly improved and reached as high as 80%. The gas drainage efficiency fell again when the roof lateral boreholes were closed and only surface vertical wells were in operation. In comparison to the initial 200-300m of retreat at LW3, gas drainage efficiency in the trial period was increased from 14%-37% up to 80%. This clearly reflects that the underground lateral boreholes achieved significantly better results than the conventional surface vertical wells used at the mine, particularly in the initial mining stage.

Figure 11 (a) shows the recorded daily longwall production delays from the commencement of LW2 to LW4. Significant production delays were seen in the initial 2-3 months of mining operation at LW2 and LW3, where surface vertical wells were used to capture goaf gases. Conversely, in LW4 where underground lateral drainage boreholes were implemented, very limited delays were incurred.

The significant reduction of longwall delays, resulting from the improved gas drainage performance, led to a remarkable increase of coal production in the initial mining stage, as shown in Figure 11 (b). The LW4 coal production, in its first two months, was increased by 79% compared to LW3.

Reduction of methane emissions to atmosphere

The trial results showed that, in addition to a significant increase in capture effectiveness with the underground lateral boreholes, the total specific gas emission was also significantly reduced for longwall operating in the same environment. This demonstrates that optimised gas drainage has reduced the raw emissions from the operation on top of increasing the abatement of emissions.

Applying the results achieved at the LW4 trial, an annual net reduction of 0.42 Mt CO2-e after drainage methane incineration could be achieved at the Blakefield South mine by adopting the drainage system trialled at LW4. This number is calculated based on LW3 gas emission data and an assumption of 315 mining operation days.
Another point worth highlighting is that the roof lateral boreholes in this particular Blakefield mining condition were analysed having no exposure to the Whybrow seam goaf gas. Gas bag samples show Whybrow goaf has a mix of about 58.8% CH$_4$, 36% N$_2$ and 5.2% CO$_2$. Excess N$_2$ level would have been seen if Whybrow goaf gases were also captured by means of gas drainage. High excess nitrogen levels are often seen in vertical goaf wells, however, no excessive N$_2$ level was seen in the underground roof lateral boreholes.

OUTCOMES AND IMPACTS

Following adoptions by Bulga

Following the successful trial at LW4, Blakefield South mine replaced the surface vertical goaf wells with underground lateral boreholes at the entire panel of LW5. The roof lateral boreholes were placed at similar locations to those at LW4, with four to five lateral boreholes intersecting the goaf during the LW retreat, as shown in Figure 12. However, a significant difference at LW5 compared to LW4 is that there were no floor lateral boreholes implemented at LW5. This difference provided a good opportunity to compare the two designs to further refine the gas drainage system for future applications.

LW5 has recently been completed. Roof lateral boreholes performed well at LW5 in capturing and controlling goaf gas and their effectiveness was similar to that of the trial at LW4. At the comparable initial mining stage, the average gas drainage flow rate and daily coal production at LW5 were 1252 l/s and 22,411 t, respectively, close to their counterparts at LW4 trial (1279 l/s and 23,121 t), and much better than that at LW3 where only vertical wells were used (1182 l/s and 13,806 t). A comparison between the LW3, LW4 trial and LW5 are shown in Table 1.

Table 1 Comparison of gas drainage and emission parameters at recent longwalls in Blakefield South mine

<table>
<thead>
<tr>
<th>LW</th>
<th>Retreat meter for comparison, m</th>
<th>Gas drainage method</th>
<th>Daily LW tones, t</th>
<th>Total gas* emission, l/s</th>
<th>Drainage gas flow rate, l/s</th>
<th>Ventilation gas flow rate, l/s</th>
<th>SGE, m3/t</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW3</td>
<td>35-223</td>
<td>Vertical</td>
<td>13,860</td>
<td>4004</td>
<td>1182</td>
<td>2822</td>
<td>25.7</td>
</tr>
<tr>
<td>LW4, Trial</td>
<td>30-259</td>
<td>Lateral (roof +floor)</td>
<td>23,121</td>
<td>2869</td>
<td>1702 (roof 1252, floor 423)</td>
<td>1167</td>
<td>10.7</td>
</tr>
<tr>
<td>LW5</td>
<td>30-220</td>
<td>Lateral (floor only)</td>
<td>22,411</td>
<td>3171</td>
<td>1252</td>
<td>1920</td>
<td>12.2</td>
</tr>
</tbody>
</table>

*Note: Gas include CO2+CH4
The consistent performance of the lateral boreholes at both LW5 and LW4 indicates that lateral boreholes are a reliable gas drainage method in such conditions as of the Blakefield South mine. In comparison to the gas drainage performance at LW3 and LW4, it has also been observed that the floor lateral boreholes in the Glen Munro seam in the LW4 trial would have made a significant contribution to controlling face and return gas levels.

The results enabled further refinement for gas drainage design for future wide applications, and have contributed to the design for the forthcoming LW7. Figure 13 shows the planned layout of LW7 goaf gas drainage lateral boreholes. It includes both roof lateral boreholes and floor lateral boreholes drilled into the Glen Munro seam. A number of floor lateral boreholes were also planned to be drilled into the lower Blakefield seam to prevent the predicted higher gas emissions in this panel compared to LW4 and LW5. To date, the borehole drilling has mostly been completed.

![Figure 13: Horizontal goaf gas drainage plan at LW7. Red and Green – roof lateral boreholes; Blue – Blakefield working section holes; Pink – Lower Blakefield seam holes; Light blue – Glen Munro seam floor holes](image)

Long term benefit from the innovative gas drainage system

According to the performance at LW4 and LW5, the innovative gas drainage system trialled at the Blakefield South mine has not only optimised methane drainage quantity and quality, but also significantly improved coal productivity and maximised methane emission abatement from mining operations. This result will deliver significant benefits to the mine in both the short and long terms. The project results have achieved the scheduled goals of this project.

The points below highlight the potential benefits the innovative gas drainage system can bring. The data are assessed on the basis of LW4 trial results.

- A net reduction of fugitive gas emissions to atmosphere of about 0.42 Mt CO2-e every year;
- An increase in methane capture and utilisation through drainage efficiency from 14-60% to around 80%, resulting in improved mining safety;
- An increase in productivity by 79% at the initial longwall mining stage through significantly reduced coal production delays; and
- An increase in efficiency delivering savings in excess of $10M per year.
CONCLUSIONS

Under the support of the Coal Mining Abatement Technology Support Package (CMATSP), a major collaboration research project between CSIRO and the Glencore Bulga Underground Operations was carried out at the Blakefield South mine. The project involves comprehensive studies covering site characterisation, integrated geotechnical and gas field monitoring and measurement, numerical modelling, and theoretical analysis. The studies have obtained many insights into the coupled strata, gas and groundwater behaviour in complex multi-seam longwall mining. These insights have resulted in a clear understanding of many factors that are critical to goaf gas drainage design, including the zone of mining influence, caving processes, gas emissions sources and emission patterns, and operational parameters. An optimal gas drainage system was therefore designed and trialled at the mine.

The trial demonstrated that the gas drainage system, which consists of a number of both roof lateral boreholes located 15-20 m above the mining seam and floor lateral boreholes in a floor gas sourcing seam, were very successful. The results showed a significant reduction in both ventilation methane level and drainage gas volume. Gas drainage efficiency was significantly improved from 14-60% to about 80% in comparison to the same section in the previously mined longwall, and gas related coal production delays were substantially reduced. As a result, coal production was increased by 79% in the trial period. It is estimated that with such a gas drainage system, the mine could reduce fugitive gas emission by 0.42 Mt CO2-e per year from its longwall operations. This innovative gas drainage system is now being used as the main goaf gas drainage method at the Blakefield South mine and has replaced the previous conventional surface goaf vertical wells system.

The project has successfully achieved the planned objectives to develop and demonstrate a holistic and optimal approach of planning, design and operational control to mine gas drainage, to maximise methane capture and to minimise fugitive emissions in gassy and multiple seam conditions. A wide adoption of this scientific approach will benefit Australia coal mines by improving mining safety, enhancing coal and methane production efficiency, and reducing fugitive methane emissions.

ACKNOWLEDGEMENTS

The authors are very grateful to the Australian Department of Industry and Science (the then Department of Resources, Energy and Tourism) for funding this research. The authors would also like to express their sincere gratitude for their significant contributions to the management and staff of both Glencore Bulga Underground Operations and CSIRO who have been involved in this project.

REFERENCES

INTEGRATED FIELD STUDIES FOR OPTIMAL GOAF GAS DRAINAGE DESIGN IN MULTI-SEAM LONGWALL MINING

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ABSTRACT: Optimisation of mine goaf gas drainage design needs to fully recognise mine geological conditions, mining induced caving characteristics and gas flow dynamics. This paper presents the integrated geotechnical and gas field studies carried out at the Blakefield South mine for optimisation of longwall goaf gas drainage. A set of instruments including extensometers, piezometers, gas pressure gauges, tracer gases and borehole cameras were adopted in these studies. The overburden strata caving processes, coal seam pore pressure changes, multi-seam gas flow paths, goaf gas pressure dynamics, and drainage borehole stability were observed. The studies enabled a clear identification of the extent and characteristics of mining influence, sources of gas emissions in both the roof and floor and goaf gas flow dynamics, and have led to a successful design and implementation of an optimal goaf gas drainage system at the Blakefield South mine.

INTRODUCTION

Longwall gas emissions in multi-seam mining conditions are very complex as they are a dynamic process associated with the interaction of mining induced strata, groundwater and gas behaviours. Previous studies (Guo et al., 2012, 2014, 2015 and Qu et al., 2015) have showed that a clear understanding of this dynamic and coupled process is critical to the development of an optimal goaf gas drainage design. In addition, goaf gas drainage performance such as drainage methane concentration is also influenced by the interaction between ventilation, gas drainage and goaf gas emission. Integrated field studies covering all of these aspects are of significance to optimisation of coal mine methane drainage and emission abatement.

Supported by the Coal Mining Abatement Technology Support Package (CMATSP), a collaborative research project, entitled “Mine methane capture optimisation and maximisation of emissions abatement” is being undertaken by CSIRO and the Glencore Bulga Underground Operations. The project aims at developing and demonstrating a holistic and optimal approach to planning, design and operational control of mine gas drainage through a better understanding of coupled strata, ground water and gas behaviour during mining.

Integrated field geotechnical and gas studies were therefore implemented at the Blakefield South mine of Bulga Underground Operations. These studies covered a series of monitoring and measurement programs including overburden movement, ground water pressure changes, seam gas migration patterns, gas content changes before and after mining, gas flow patterns in the goaf, and surface vertical drainage borehole stability. This paper introduces the integrated field studies and presents the key results.

MINE CONDITIONS

The Blakefield South mine is located 15 km southwest of Singleton in New South Wales. It lies in the Hunter Coalfield, Northern Sydney Basin. The strata associated with the coal seams were laid down

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during the Permian Period and comprise the Wollombi and Wittingham Coal Measures of the Singleton Supergroup. The Wollombi Coal Measures are stratigraphically higher and contain poor quality coal seams while the Wittingham Coal Measures contain better quality and thicker coal seams.

The Blakefield South mine uses a three heading gate road development system to access longwall panels, which are 400 m wide (rib-to-rib) and up to 3.5 km long. Figure 1 shows the mine plan as of September 2013. The longwalls (bright yellow lines) are operated underneath existent goafs (dark yellow lines), mined by the South Bulga Mine in 1994-2003. The South Bulga mine extracted coal from the lower Whybrow coal seam, which lies approximately 70-100 m above the current mining seam. Panels in the Whybrow longwall were about 250 m wide with pillars between panels at 25 m.

![Figure 1: Mine plan of The Blakefield South mine as of September 2013](image)

The mining seam, Blakefield, comprises four coal plies, 300, 301, 302 and 303 while the longwall extracts coal from the top three plies with a cutting height ranging from 2.8 m to 3.4 m. The Blakefield seam generally dips from north to south at 3 degrees, and its depth of cover above LW1 to LW3 varies between approximately 130 m and 260 m.

The Blakefield seam is surrounded by a number of gassy coal seams. It is overlain by the Redbank Creek, Wambo and Whynot seams, and underlain by the Glen Munro and Woodlands Hill seams. All these coal seams were predicted to be gas emission sources according to the FLugge model. Figure 2 shows a cross section of the Blakefield South mine with the gas content of each coal seam marked.

**INTEGRATED FIELD MONITORING AND MEASUREMENT PROGRAMS**

Prior to the development of the integrated field studies, extensive site characterisation of in-situ strata, hydrogeological and gas conditions were carried out at the Blakefield South mine. Furthermore, longwall gas drainage technical and operational data were collected and reviewed. It was decided that the following key aspects needed to be studied in order to provide adequate information to optimise gas drainage design for this specific mine.

- Caving process, extent and magnitude under the existing overlying Whybrow goafs
• coal seam pore pressure changes in response to mining
• gas emission sources, patterns, and dynamics as longwall advances
• effect of the existing Whybrow goafs on gas drainage performance
• surface goaf vertical well stability
• interaction between gas drainage, ventilation, and barometer pressure.

To study the above aspects, integrated monitoring and measurement programs were developed and implemented at the Blakefield South mine LW3. Figure 3 shows the plan view of the monitoring instrument locations. The key monitoring programs are described below.

![Figure 2: Cross section of the Blakefield South mine](image)

![Figure 3: Plan of the integrated monitoring programs](image)

**Surface deep-hole multi-anchor extensometers**

Two surface extensometers were installed, with one located at 142 m to the longwall panel maingate (MG) edge and the other 123 m to the tailgate (TG) edge. Each extensometer consisted of 11 anchors...
covering a depth range from 11 m above the mining seam to 10 m below the ground surface. Most of the anchors were placed in the interburden between the Whybrow goaf and the working section. Both extensometer boreholes were cased to about 10-15 m below the Whybrow goaf to maintain borehole integrity. A surface data logger system, equipped with a solar panel, was installed to continuously record the readings at 15 minute intervals. Figure 4 (a) shows one of the extensometers surface data logger systems.

**Surface deep-hole multi-sensor piezometers**

Several piezometers were installed within the project period to monitor coal seam pore pressure dynamic changes with longwall mining and to assess gas desorption and migration processes in various coal seams. A key instrument was P3 (as denoted in Figure 3), which was installed at the MG chain pillar zone with a distance of 52 m from MG edge. This location was selected to avoid cable failure which might be caused by strata movement and surface subsidence. A total of seven piezometers were installed in this hole into all major coal seams from Redbank Creek to Woodlands Hill, as well as into interburden between the coal seams. Fibre optic piezometers were selected for this hole to avoid the risks of lightning strike. The piezometer readings were continuously recorded at 15 minute intervals by a surface data logger, as shown in Figure 4 (b).

**Goaf gas pressure dynamic changes using pressure gauges and tubes**

The mine uses a force-exhaust ventilation system to minimise pressure difference between the Blakefield South workings compared to the Whybrow old workings and to the surface. Monitoring goaf gas pressure dynamics is critical to assessing the effect of this ventilation system on the control of goaf gas emissions as well as goaf gas drainage performance. It is also of significance to understand the dynamic goaf gas pressure environment for development of an optimal gas drainage system.

Goaf gas pressure monitoring was achieved by connecting a tube between the goaf and a pressure gauge sitting on the ground surface to sense the variation of differential pressure between the goaf and surface atmosphere. Monitoring locations of goaf gas pressure changes included the Whybrow goaf, the Wambo seam and a few locations on the MG side of the active LW3 goaf. The surface pressure monitoring system (Figure 4(c)) was connected to the mine’s monitoring system Citec and the pressure gauge monitoring data was continuously recorded.

**Tracing seam and goaf gas flows**

Tracer gases were released from various locations of LW3 to study gas flows in the overburden fractured strata and the goaf. These tests included the injections of Sulfur hexafluoride (SF$_6$) into the Whybrow goafs, injection of SF6 into a packer-isolated borehole section at the Wambo seam, injections of helium into the LW3 intake corner, and injection of helium into the LW3 MG intake-goaf corner as well as into the LW3 deep goaf. Gas samples were taken through the mine’s tube bundle system from various designated locations in the LW3 goaf, ventilation return, and surface vertical goaf drainage wells. Gas composition measurements were taken with the mine’s Gas Chromatography for helium and CSIRO’s Gas Chromatography for SF$_6$.

The tracer tests were to understand gas flow characteristics and paths within the flow field between the Whybrow goafs, overburden fractured strata, surrounding coal seams, Blakefield goafs, and the ventilation return. The tests also helped assess the effectiveness and mechanisms of goaf gas drainage.

**Coal seam gas content before and after LW3 excavation**

Measurement of coal seam gas content before and after longwall extraction directly identifies gas emission sources and evaluates gas release levels from various coal seams during longwall operation. Gas content measurements at different locations above the longwall panel can also reveal the emission
patterns around the longwall panel. Literature review shows that very limited data of gas content measurement before and after mining are available.

Gas content measurements were carried out at three locations of LW3, approximately along a line across the panel width. The two extensometers boreholes, E1 and E2, and the piezometer borehole P3 were drilled with coal cores taken from various coal seams for pre-mining gas content measurement. Three other holes were drilled after LW3 was finished at locations a few meters away from each of the pre-mining gas content measurement boreholes.

Surface vertical goaf drainage wells inspection using special cameras

The surface vertical goaf drainage wells at the Blakefield South mine were not satisfactory in controlling goaf gas emissions, particularly in the initial 400 m from the longwall start-up. Some drainage wells were operated for only a few days and then closed due to unacceptable high levels of oxygen in the drainage gas. In addition, the surface vertical wells were observed as having limited effect on reducing ventilation methane levels despite the high drainage flow rate in the wells. It was therefore valuable to inspect the drainage well integrity and stability status to further analyse issues associated with the poor performance of the surface vertical wells. The inspection results were also expected to be critical to the study of vertical well gas drainage mechanisms.

The inspection was conducted using SIMTAS special borehole cameras. A total of ten surface vertical wells at both LW3 and LW2 were inspected after LW3 was completed. These inspected boreholes were located at various locations relative to both the LW3 goaf edges and Whybrow goaf edges, with an aim of comparing the effect of drainage well locations on their stability and integrity.

![Figures](image1.png)

**Figure 4: Some field monitoring systems implemented at the Blakefield South mine**

KEY RESULTS AND ANALYSIS

Overburden caving process and features

Both the extensometers successfully obtained the dynamic movement of overburden at different levels versus time. It is noted that the E1 gave more typical results than E2 as the former was located in the mid Whybrow goaf zone while the latter was located on the edge of Whybrow pillar zone. Figure 5 shows the monitored overburden movement by the extensometer E1.

Key observations from Figure 5 include:

- Strata movement at a level 11 m above the mining seam started at 12 m inbye the longwall face, and mining induced fractures and delaminations extended up to the Redbank Creek seam at 31 m inbye of the longwall face. This shows a quick response of overburden movement and relaxation as a result of mining which can lead to a quick release of coal seam gas.
The dynamic process of overburden movement along the longwall retreat direction was clear. The overburden moved quickly once being inbye of the longwall face, reached the maximum delamination at about 60 m inbye, and became stable in another 60 m of retreat.

- Significant delaminations occurred through and up to the Whybrow goaf (as shown in Figure 5 (b)), indicating a high horizontal permeability was present in the goaf and the interburden.
- According to the typical model of overburden deformation (Peng, 2006), the overburden from 11 m up to the Redbank Creek seam could be characterised as the fractured zone as the delamination characteristics through the zone were similar.

![Figure 5: Monitored overburden movement by extensometer E1](image)

**Coal seam pore pressure changes**

The surface piezometer P3 was installed at the pillar zone between LW3 and LW4 with a distance of 52 m from LW3 MG goaf edge. Figure 6 shows the monitored pore pressure changes in all the major coal seams with longwall advance. Gas emission sources were also estimated from the monitored pore pressure changes.

The following key observations can be drawn from Figure 6:

- Redbank Creek, Wambo, lower Blakefield and Glen Munro coal seams were significantly depressurised during mining, indicating these coal seams would release gases and therefore are drainage targets.
- Woodlands Hill had no reduction of pore pressure, indicating gas would not be desorbed and released.
- The trend of pore pressure changes among the coal seams were different, indicating different gas desorption and release characteristics. Pore pressures in the two overlying coal seams at a location 500 m inbye of the longwall face were very low (close to atmospheric pressure); adsorbed gases in these coal seams would have mostly been released.
- Coal seam pore pressure started to decline at least 50m outbye of the longwall face, and continued up to 250 m inbye of the longwall face in the case of the Wambo seam.

With the measured concentration and the mine’s monitored flow rate, the quantity of SF6 travelling through drainage vertical wells and the ventilation return could be calculated and thus the SF6 flows...
among various paths could be characterised. For tracing tests with helium, however, only major flow paths were able to be characterised due to the impact of background helium level in the goaf.

Figure 6: Coal seam pressure changes as longwall advances

Key observations associated with goaf gas drainage design are highlighted below.

- No SF6 was detected in the LW3 active goaf and in the ventilation return after the tracer was released into the overlying Whybrow goaf. It can be concluded that the Whybrow goaf gases did not flow down to the active goaf and therefore contribute to the ventilation gas makes. In other words, capturing gas from Whybrow goafs did not have an effect on reducing ventilation gas levels.

- High interconnection or conductivity exists within and across the overlying Whybrow goafs. This is indicated by the detection of SF6 from almost all operating drainage wells after releasing SF6 into a Whybrow goaf.

- Almost half of the SF6 released into the Wambo seam section flowed through the ventilation return and another half was captured by vertical drainage wells. This, to some extent, reflects the effectiveness of vertical drainage wells in capturing and controlling the coal seam roof gases.

- Deep goaf gases (745 m inbye of the longwall face) could hardly migrate into the ventilation return.

Figure 7: Gas flow characteristics analysed from a tracer test injecting SF6 into Wambo seam

Goaf gas pressure dynamics

Figure 8 shows the monitored gas pressure changes in the Wambo seam section versus methane levels in the ventilation return, for a period of four days. Gas pressure was monitored through a tube
connecting the isolated Wambo seam section of a borehole to a pressure gauge sitting on the surface. Under the force-exhaust ventilation system, pressures in the active LW3 goaf and ventilation circuit were close to that on the surface. Therefore, the monitored Wambo seam gas pressure also reflects the differential pressure between Wambo seam and the active LW3 goaf.

It can be concluded from Figure 8 that:

- Changes of differential pressure between the Wambo seam and the active goaf correlate well with the changes of methane levels in the ventilation return. Increased differential pressure caused increased methane levels in the ventilation return simultaneously.
- The good correlation suggests that the dynamic barometric pressure change is the primary driver of the variation of the ventilation gas makes. It further indicates that an effective gas drainage system under such circumstances should have a sufficient capacity to be able to control such goaf gas pressure changes.

**Figure 8: Correlation between monitored Wambo seam differential pressure versus methane levels in the ventilation return**

**Gas content before and after mining**

Table 1 presents the measured gas contents of various coal seams before and after LW3 excavation at the three measurement locations shown in Figure 9. Key observations are:

- Redbank Creek, Wambo, and Glen Munro seams are obviously significant gas emission sources, while the Woodlands Hill seam is not.
- Special attention needs to be paid to gas emissions from the underlying Glen Munro seam as part of the gas emission most likely to contribute to the gas make in the ventilation return.
- The gas content before mining at various locations across the LW3 panel width are different and appear lower at locations closer to the mined LW2 panel. This suggests that the LW2 excavation made an impact on the gas content distribution of the LW3 panel.
- The extent of gas release from neighboring coal seams extends beyond the direct mining vicinity and into the unmined side area. Coal seams above the unmined side chain-pillar also released a significant percentage of gas.
Figure 9: Locations of gas content measurement boreholes

Table 1: Gas content measurement results before and after the LW3 mining

<table>
<thead>
<tr>
<th>Coal seams</th>
<th>SBD205 zone</th>
<th>SBD209 zone</th>
<th>SBD211 zone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pre-mining, m3/t</td>
<td>Post-mining, m3/t</td>
<td>Gas released, m3/t</td>
</tr>
<tr>
<td>Redbank Creek</td>
<td>3.6</td>
<td>1.92</td>
<td>1.68</td>
</tr>
<tr>
<td>Wambo</td>
<td>5.99</td>
<td>2.87</td>
<td>3.12</td>
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<tr>
<td>Blakefield Upper</td>
<td>1.19</td>
<td>1.16</td>
<td>-0.02</td>
</tr>
<tr>
<td>Blakefield Lower</td>
<td>1.03</td>
<td>1.05</td>
<td>0.02</td>
</tr>
<tr>
<td>Glen Munro</td>
<td>6.27</td>
<td>4.02</td>
<td>2.25</td>
</tr>
<tr>
<td>Woodlands hill</td>
<td>13.345</td>
<td>13.8</td>
<td>-0.455</td>
</tr>
</tbody>
</table>

*Note: SBD209 hole was located only a few meters away from the surface vertical drainage well 3L.

Figure 10: Different types of borehole blockages inspected by a borehole camera

Effect of drainage borehole location and operational parameter on gas drainage performance

Vertical drainage wells at The Blakefield South LW3 were located at two typical locations: close to the TG (30 m to 80 m offset TG) and in the middle of the longwall block. To assess the drainage performances at the two locations, the total captured seam gas volume of vertical wells at various locations was compared, as shown in Figure 11 (a). In addition, the relationship between suction pressure and gas flow rate of two nearby vertical wells 3H (located in the TG side) and 3J (located in the mid-block) were compared as shown in Figure 11 (b).
Key observations were:

- Vertical wells located in the TG sides of the longwall generally captured more gas than those located in the mid block.

- Under the same suction pressure, the TG side well 3H had about 70% higher flow rate than the mid-block well 3J. This indicates that gas drainage wells located in the TG side of the longwall have better connectivity to rich gases in the goaf and therefore capture more gases.

![Diagram](image1.png) ![Diagram](image2.png)

(a) Total drainage gas volume versus vertical well location (b) Relationship between drainage well suction pressure and gas flow rate (3H: TG side, 3J: mid-block)

**Figure 11: Effect of vertical well location and applied suction pressures on drainage flow**

**CONCLUSIONS**

Supported by the Government CMATSP project “Mine methane capture optimisation and maximisation of emissions abatement”, integrated field studies were designed and implemented at the complex Blakefield South mine to study the coupled strata, ground water and gas behaviours. The objective of these field studies was to provide critical information to develop an optimal longwall goaf gas drainage design.

The integrated field studies covered a number of aspects of mining induced strata behaviour including overburden movement with extensometers, coal seam pore pressure with piezometers, seam gas migration patterns with tracer tests, seam gas content measurement before and after mining, borehole stability inspection, and goaf gas pressure dynamics. These monitoring programs successfully provided insights into the coupled strata, ground water and gas responses to mining. Critical information for gas drainage design, including gas emission sources, zones of strata caving and fracturing, extent of mining influence, multi-seam gas migration patterns and surface goaf well stability, were obtained.

The results have led to an innovative gas drainage design which was trialled successfully at the Blakefield South mine.

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The authors are very grateful to the Australian Department of Industry and Science (the then Department of Resources, Energy and Tourism) for funding this research. The authors would also like to express their sincere gratitude to the management and staff of Glencore Bulga Underground Operations for their great support on this project, and their CSIRO colleagues Dr Jun-Seok Bae, Dr Xianchun Li and Dr Xin Yu for their contributions to the tracer gas tests.

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OPERATIONAL CONSIDERATIONS FOR TUBE BUNDLE GAS MONITORING SYSTEMS

Martin Watkinson¹, Snezana Bajic², Lauren Forrester³ and Larry Rayn⁴

ABSTRACT: Tube bundle gas monitoring systems are now common practice in the Australian underground coal mining industry. Systems are in operation at all underground coal mines in Queensland, all longwall coal mines in New South Wales, and most bord and pillar coal mines in New South Wales. This paper utilises the information assessed by Simtars as part of the project “The Application of Tube Bundle Systems in the Prevention of Mine Fires and Explosions and Post-Event Response”, prepared for the National Institute for Occupational Safety and Health (NIOSH), under contract number 200-2013-56949. The operational considerations and decisions required to ensure the optimum operation of the system will be outlined. Installation, maintenance, training requirements, alarm settings and interpretation of the gas results will be taken into account. Practical solutions and explanations will be provided, drawing on Australian and overseas experience, as well as information publicised by the National Coal Board in the United Kingdom and the United States Bureau of Mines (USBM).

INTRODUCTION

Simtars conducted an investigation called “The Application of Tube Bundle Systems in the Prevention of Mine Fires and Explosions and Post-Event Response”, prepared for the National Institute for Occupational Safety and Health (NIOSH), under contract number 200-2013-56949. This work involved a review of the history of the development of tube bundle systems and covers any legislative requirements and practical aspects of operating tube bundle systems. This paper utilises the information that was collected as a result of that investigation as well as the operational experiences of Simtars staff. The paper discusses the operational requirements for tube bundle installations, system maintenance requirements and provides recommendations for the optimisation of site systems. Training requirements for maintenance and software operation are also presented, including the review of alarm settings with the utilisation of valid trigger action response plans.

Early developments and background

Chamberlain (1970) describes the early introduction of tube bundle systems for the monitoring of longwall goaves (gobs) on advancing longwalls in the United Kingdom (UK). He identified that the technology was developed in Germany by Dr Luft of Bergbau-Forschung Essen and the equipment was manufactured by Maihak. The initial use of the systems was in an underground application where the analyser was housed in a flame proof enclosure and the results telemetered to the surface through colliery communication systems. These systems had been used in the Saar region of Germany and in Scotland.

Detection is based on the principle that gases such as carbon monoxide (CO), carbon dioxide (CO2) and methane (CH4) all absorb certain wavelengths of infrared radiation (Chamberlain 1970). This is not the case for gases such as oxygen and nitrogen. It is the bond between the different elements which determines the ability of the gas to absorb the radiation. Oxygen (O2) is however detectable by a

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paramagnetic analyser, these again were developed in the 1960’s and were being considered for tube bundle system installations in the UK in the 1970’s. Nitrogen can only be detected using gas chromatography.

The development of tube bundles for hydraulic controls enabled the development of the tube bundle gas monitoring systems we know today. A bundle of tubes is taken underground through a shaft, a borehole or a drift then distributed to the required monitoring points. This enabled the ongoing monitoring of the underground environment when the power was isolated at 1.25% CH4.

Chamberlain (1970) states that a tube bundle installation in the Durham area of the National Coal Board (NCB) was being used to take samples directly from the goaf and identifies that if the samples analysed are high in methane the carbon monoxide can be calculated on an oxygen free-methane free basis. Chamberlain also mentions the advantage of the system allowing gas chromatography to be undertaken.

OPERATIONAL CONSIDERATIONS

The National Coal Board (NCB) in the UK produced a handbook which was updated in 1974 (NCB 1974), it was developed to provide the operational guidelines for colliery personnel for the installation and operation of tube bundle systems. The foreword indicated that the handbook contained only guidelines and not rules as the technique was still rapidly developing. “The emphasis was changed to make it suitable for a wider audience who should now include not only those who have to install maintain or use it, but also those with only a temporary or limited interest in it or with a specialised knowledge of part of it such as suppliers, senior management stores officers.” (NCB 1974) The handbook still provides an excellent source of information relating to tube bundles and remains a very good “how to and why” guide. It was noted that the advantage of tube bundle systems was the simplicity of the underground equipment with analysing and controlling hardware on the surface, without restrictions associated with electrical equipment being underground.

The United State Bureau of Mines (USBM) prepared a circular similar to the NCB handbook on tube bundle systems. The circular was prepared by Litton (1983) and it was targeted at the design criteria for tube bundle systems. The circular also states that “pneumatic monitoring systems can be used to monitor for hazards that develop quite quickly”.

Installation

When considering options for the installation of a tube bundle system, borehole access (when available) provides the potential to reduce the length of the tubes deployed and hence reduce the delay time for the gas to reach the analyser. It is recommended that 12.7 mm (0.5 inch) Outer Diameter (OD) tube is used for runs up to 6 km and thereafter 15.9 mm (5/8 inch) OD tubing is used. In cold climates it is necessary to insulate, and in some cases heat the tubes to prevent condensation and freezing as the tubes exit the borehole.

It is also prudent to install water trap arrangements at the bottom of the borehole/shaft or drift at the low point where water will accumulate. A maintenance regime needs to be in place for the planned emptying of these water traps. In extreme cases large accumulation vessels may be required, ie in very cold climates such as in Canada, China or USA.

Groups of 5 or 10 tubes are connected to an individual purge pump (the pump that keeps the gas samples flowing in the tubes at all times). Some suppliers do not offer the option of individual line regulation but advise that all tubes of similar length are connected to the same purge pump to equalise the flow. This is not practicable in all mining situations and it is therefore recommended that individual regulators are fitted to all tubes to balance the flows for optimum performance of the system. It is also recommended that if a mine has a system that does not have this functionality that the mine investigates
the option with the supplier of retrofitting line regulators. The other individual lines can then be balanced to ensure the same flow rate in each of the tubes, i.e. set the longest tube to zero regulation and balance all the others to that setting. A typical schematic of the surface installation is shown at Figure 1.

Other operational considerations are to run the tubes in the return roadways to avoid unnecessary condensation. Most mines have the tube run in the intake because this facilitates easier access for installation and inspection. It is worthwhile considering the operational aspect of running them in the return from the point of reduced condensation and the issues of emptying water traps. The tubes should be secured tightly every 2 m to the roof or rib, and accumulations of slack or sagging tube avoided, this will help protect the tubes from damage. Tube sagging can become a collection point for moisture and is also not as effective at surviving any possible underground explosion (Brady et al., 2015).

As the tubing is food grade polyethylene, they are prone to deteriorating when exposed to ultra violet (UV) light. Even bundles of tubes which are exposed to UV light have been seen in a deteriorated state with the inner coloured tubes brittle and easy to break. If tubes have to be run over land it is prudent to run them in trenches, culverts or pipes where they will not be exposed to UV radiation.

It is imperative that good quality tubing and joiners are purchased. There have been occasions when cheaper tubing, poor quality joiners and poor installation of joiners have compromised the installation by causing leaks that were difficult to eliminate.

AS2290 Part 3 defines the leakage testing required for a tube bundle system (Standards Australia 1990). This does not include the pressure testing of tubes. The standard required the introduction of a known gas mixture at the inbye end of the tubes and it also defines the pass criteria on the gaseous result. One of the tube bundle equipment suppliers is now offering a service to leak-test tubes. The approach used is to pass nitrogen down the tube from the surface, then the flow is reversed up the tube and the gas analysed on an oxygen analyser. As the flow rate is known and the length of the tube is also know, it is claimed that leaks can be detected within ±20 m. AS2290 Part 3 is currently undergoing a review and update by industry experts.

The final point to note is the adequate sample preparation for the gas before it enters the analyser. The preparation involves chilling the gas sample to 4°C in order to condense out the water vapour. Many of the systems installed in Australia contain a large amount of stainless steel tubing and it is important to note the importance of proper air temperature in the room, in order to avoid issues with condensation within the tubing.

**Tube bundle monitoring software**

The Tube bundle software needs to meet the requirements of a group of professionals including Ventilation Officers, Control Room Operators (CROs), Control Engineers and Electricians. A common approach for visualisation is to have the mine plan and gas results as shown at Figure 2.

**User authentication**

Each individual user needs to have a unique login to the tube bundle software to ensure system configuration changes are reasonable and approved (for example gas alarm thresholds). The software would allow for different levels of access:

1. **Operator:** alarm acknowledgement, trending, holding on a tube for continual monitoring.
2. **Maintenance:** the same access as the operator with the additional rights to calibrate the analyser and troubleshoot via the maintenance screen.
3. **Administrator:** the same access as the maintenance with the additional rights to change all the configuration settings and alarm setting.
Alarms

The tube bundle software will generate alarms for all the gases being monitored in at least four levels ie HHPV (Rising Alarm Level), HPV (Rising Warning Level), LPV (Falling Warning Level), and LLPV (Falling Warning Level). The four alarm levels allow for the notification that the gas value has risen or fallen away from analyser baseline value. For gases such as CO, CO2 and CH4 the LPV and LLPV alarms are used mainly for diagnostic purposes and the same would apply for increasing O2 value (for example, O2 greater than 22.5% is not possible in fresh air). The diagnostic gas thresholds can be set reasonably tight on the fresh air point to monitor the analysers drift and raise an alarm if a calibration is required.

Important alarms should have a procedure for the Control Room Operators (CRO) to follow. The same principle would apply for tube bundle specific hardware alarms, for example purge pump failure.

An acknowledged alarm does not mean that the alarm condition has dissipated, it simply means that the CRO has seen the alarm and has accepted responsibility for it. Unfortunately, a mine site control room can be a very busy place and the work undertaken can change dynamically. Hence, it is possible that a high level alarm can be buried or hidden among a number of low level alarms. A solution to this problem is to have an acknowledge timeout set for the high level alarms where the alarm will reactivate after set period of time (for example 60 minute delay), in order to alert the operator that the alarm condition is still present and in fact could be worse.

Calibration and troubleshooting

The calibration of the tube bundle analyser is a critical part of maintaining the gas monitoring at a high standard. The process to calibrate the analyser needs to be easy and relatively fast.

The maintenance screen should allow for calibration and troubleshooting of the tube bundle system. While the calibration of the analyser can be streamlined, the troubleshooting of incorrect gas results requires the manual operation of the various valves in order to identify the problem. For example, high CH4 in a relatively fresh air return could be caused by cross contamination from a previous goaf tube.

Important parameter calculation and configuration

At a minimum to meet the Queensland Mining Regulations, Graham’s ratio and CO/CO2 ratio need to be automatically calculated and alarms established. In addition, calculation of CO make can also be another useful tool for the early detection of spontaneous combustion.

Graham’s ratio and the CO/CO2 ratio should be configured to have an initial location and final location. The tube bundle software needs to have the functionality to setup the necessary initial requirements, or if no initial point is available the option to select a fresh air reference point within the mine, or alternatively use valid fresh air readings.

If a large storm was to move over a mine site the barometric pressure could fall quite sharply. Atmospheric pressure can influence the amount of gas being released from both the active and the sealed goaf/gob. The barometric rate of change is an alarm that monitors the change in the barometric pressure and warns the CRO that the goaves may start to rapidly breathe out.

Data storage

The tube bundle gas results need to be recorded, continually archived and backed up to a secure storage server. The gas data may be required years in the future and easy retrieval is very important. The gas data needs to be backed up to a secure storage server to prevent data loss from failed hard drives, lightning damage and server theft.
All the alarms need to be logged, including the threshold of the value that was exceeded, for the following reasons:

- the alarms can be reviewed to ensure the alarm thresholds are reasonable.
- the alarm thresholds need to be set to identify a movement away from the baseline, without generating a large number of not so important alarms due to barometric changes.
- the number of alarms needed be minimised to prevent CRO alarm fatigue where every alarm is acknowledged. In this case the alarm procedure is not followed and there is a safety risk that important alarms are ignored.

The software needs to record all the changes to the system configuration for the following reasons:

1. If there are any problems with the new setting, the previous values can be reinstated.
2. The administrator who made the changes can be identified and the reason for the change confirmed.

**Tube bundle system diagnostics**

The tube bundle system should monitor fresh air as a simple diagnostic check. If the tube bundle system cannot successfully monitor fresh air then concerns need to be raised as to the monitoring of the gaseous environment underground.

If the tube bundle system is located inside a closed building then there is potential for a build up of gas. Hence the tube bundle system should monitor the air inside the building to ensure that there are no gas leaks. A risk assessment and various controls would need to be undertaken to ensure that if a gas leak is detected that the power to the tube bundle system is disconnected prior to the point of reaching explosive gas mixture.

Each tube should have its vacuum and sample flow monitored and alarmed on to assist with troubleshooting of any tube issues i.e. damaged tubes or leaking unions.

Any critical equipment for the functionality of the tube bundle system, ie purge pumps need to be monitored and alarmed on when their operational status moves outside the normal parameters.

**Maintenance**

**System support**

The maintenance, calibration and integrity testing of a tube bundle system can be a challenging task, it is highly recommended that a strong robust relationship is established between the tube bundle hardware suppliers, tube bundle software suppliers and the mine site personnel. It would be an advantage if the mine site were to have an electrician or Deputy specially trained to support and maintain the tube bundle system. The monthly tube integrity testing, as required by Australia Standard 2290 Part 3, is a major part of the maintenance of the tube bundle system. The tube integrity testing is required for the following reasons:

1. To confirm that the tube monitoring location is correctly identified in the software. Unfortunately, the tubes are frequently cut, extended and amended with different colours many times, making it quite easy for a tube to become mixed up with a different tube, especially if both are sampling a similar atmosphere.

2. To verify that the tube does not have any leaks. The tubes are under ~60kPa of vacuum which means that any small leak will dilute the sample before it reaches the surface. While some leaks are very obvious, for example a goaf tube going to fresh air, if a return tube was to have a leak it
would not be so obvious. If the mine has an incident and the power underground is turned off due to high CH4, the only monitoring available for the decision making process on re-entry would be via the tube bundle system (or sampling from bore holes). Hence it is imperative that the tubes are collecting a representative sample correctly.

3. To calculate the tube delay time between the sample collection and when the sample is analysed on the surface by the gas analyser. If a high CO alarm is raised on a tube, the CRO needs to know when the sample was taken underground in order to determine if the condition underground could have deteriorated further.

As the tube bundle system is needed to operate 24/7, a list of critical spare parts should be formulated and stored onsite. The critical spares form two categories, namely regular maintenance spare parts and general spares. The regular maintenance spare parts include the various filters, dryer parts and membranes. The general spares include the gas analyser, Programmable Logic Controller (PLC), sample and purge pumps, and various valves. It is recommended that all the spare parts are the same as the Original Equipment Manufacturer OEM supplied.

In addition, the tube bundle system server and control room workstation need to have a replacement plan in place to ensure that the system can maintain uptime after server/computer failures.

Training

Gas monitoring and interpretation software has many configuration settings, and it takes time to become competent in using the software. In addition, even when the mines Ventilation Officer (VO) has set all requirements (the gas alarm thresholds, gas ratio initial conditions, tube bundle lag times), the mining condition can change and require tubes to be relocated, resulting in the settings needing to be updated again. This is a continuous process in underground coal mining. Each time the tubes are relocated, their sampling location needs to be carefully updated. This will ensure that if there was an emergency situation, the exact location of the gas sample source is known, for example inbye or outbye of trapped mine workers. Each month during the tube integrity testing process, the tube delay times need to be verified to reflect actual delay between the sample collection and the alarm condition identified on the surface.

Most of the configuration and updating of the gas monitoring software is undertaken by the VO. One issue to consider is frequent staff movement from mine to mine, particularly the VO. If the VO were to move to another mine site, the task of maintaining the gas monitoring software will be taken on by a new VO, who may not have had the adequate OEM training and only has a rudimentary understanding of the software. Unfortunately, while some software can appear to be simple to use, the requirements of some configuration changes can be complex, for example the setting of the initial conditions for the gas ratios and the rate of change.

Staff turnover in the control room is another area where ongoing training is required on the gas monitoring software. This could improve the gas monitoring and decision making process early in an emergency situation.

Ideally the gas monitoring software should lead itself to be simulated so trial emergencies/desk top simulations can be conducted in a safe manner. Simulation software is used for the Queensland Level 1 Emergency Exercise. While only one Queensland mine a year has the Level 1 exercise, each mine conducts a Level 2 exercise and when simulated gas data is used, it contributes to the realism of the exercise.
Gas interpretation software

Once the gas data is collected by the tube bundle software, the gas information can be reviewed in the gas interpretation software. The gas data is generally displayed for review in the manner below:

1. The raw gas data or equations can be displayed on a time graph to show its trend over time. Figure 3 is showing CH4, CO, O2 and CO2 gas concentration in seal 512 in time. The data used is from Moura No. 2, before and after the explosion event on 7th August 1994.

2. In order to display the gas sample’s combustible nature, a Coward Triangle is a useful display. Figure 4 shows the position of gas mixture at point 512 seal (Moura No. 2) on a Coward Triangle. The Coward Triangle shows the percentage of oxygen with the total percentage of flammable gases in the gas mixture that was analysed. There are four zones on the triangle: explosive, fuel lean, fuel rich and the non-explosive zone. The point plotted on a triangle indicates the status of the gas atmosphere analysed at the time.

3. An Ellicott diagram is suited for trending purposes. This is useful for predicting if an atmosphere is trending towards the explosive range. Figure 5 shows the Ellicott Diagram for seal 512 on 7th August 1994, with clear indication that the atmosphere analysed is trending towards the explosive zone.

The gas interpretation software also needs to be flexible enough to allow the trending of custom equations that have been tailored to meet a specific task.

DATA INTERPRETATION

Trigger Action Response Plans (TARPs) are used to define what are normal and abnormal underground conditions, and the appropriate actions that should be undertaken should certain levels be triggered. TARPs generally consist of three or four levels ranging from normal conditions (level 1) to abnormal conditions requiring evacuation of personnel (level three or four). The data produced by a mines tube bundle system often forms the basis for the triggering of the different levels in the spontaneous combustion TARP. The quality of the installation and maintenance of a tube bundle system is therefore a direct influence on the quality and validity of the data produced. Poorly maintained installations may result in false alarms being raised, leading to complacency by mine personnel. Complacency and failure to acknowledge the danger posed by alarms can have fatal consequences.

Cliff (2009) listed eight fundamental principles for TARPs:

- They must be simple and robust
- The TARPs must be adequately resourced both in terms of personnel and equipment
- The focus of TARPs should be on prevention and control through early detection
- Setting triggers requires detailed knowledge of what is normal
- TARPs need to be regularly reviewed and revised as necessary and as experience dictates
- There is no substitute for high quality mine environment monitoring systems
- TARPs should be set based on the best available advice – both on site and off site
- If a TARP mandates an action, then that action must be carried out, properly and promptly

Trigger levels for use in TARPs should not be set using literature values, the values need to be derived from the mines own historical data with a sound scientific basis underpinning them. In the case of a new mine with no historical data, consideration needs to be given to laboratory testing results for the mines coal, mining factors and data from nearby mines if available and applicable. Caution needs to be used when using data from neighbouring mines to validate acceptable trigger levels for new mines. There are cases where neighbouring mines have a great deal of variation in the seam gas content, such as a methane based seam gas at one mine and a mixture of methane and carbon dioxide seam gas at the mine next door. A CO make in excess of 100L/min may be normal for one mine, whereas a CO make of around 5L/min may be normal for another. A basis for what constitutes normal at the mine needs to be established before higher trigger levels can be formulated. Normal conditions can vary greatly between
different seams, the location of the mine (ie NSW -Sydney Basin or QLD - Bowen Basin), the seam depth that the mine is targeting, different mining methods and even the inherent properties of the coal being mined (Cliff et al., 2014).

The approved standard for the use of gas monitoring systems QMD 96 7398 recommended that CO make be used for carbon monoxide related gas alarms (Queensland Department of Mines and Energy 1996). Whilst this standard was superseded by the recognised standard, this recommendation is still valid depending on the application of the monitoring location. Monitoring locations that have no airflow (eg goaf seals) should use the raw value, whereas monitoring locations that do have an airflow (eg returns) should use CO make. Ventilation changes could have the effect of diluting a raw carbon monoxide value, and therefore potentially masking a heating. Changes to ventilation do not have the same effect on CO make as the volume of carbon monoxide does not change (Cliff et al., 2014).

Stephan (2000) describes the use of statistical methods when setting trigger levels. Data should be collected from an appropriate location, ensuring that there is sufficient data and that it is not compromised by outliers. Outliers could be caused by calibration of sensors or integrity testing of tubes as an example. A histogram of the data should be plotted and analysed to ensure that a normal distribution is apparent. Insufficient data points could result in the data not being normally distributed. Once a normal distribution has been established, Stephan recommends the use of three standard deviations above the mean of the data set as the point to set the first trigger level. Caution must also be given to data that has additional influences such as carbon monoxide being produced by diesel vehicles. This type of influence may result in the data not having a true normal distribution. Stephan has provided an example of how a statistical approach can be utilised when setting trigger levels, thus providing one aspect of sound scientific basis for the justification of the trigger level values.

The rate of change is an important factor that needs to be further investigated for its possible incorporation into TARPs, or at the very least its use as a tool in the decision making process for setting trigger levels in TARPs. Small and large scale testing has demonstrated that there is a rate of change for common indicators such as carbon monoxide as the temperature of the coal increases, with a final exponential increase once thermal runaway is achieved (Clarkson 2005). Incorporation of the rate of change may provide a crucial missing link for mine personnel when faced with the dilemma of where to set the trigger levels for a TARP. Similar to the approach made by Stephan (2000) with the use of statistical analysis, the rate of change has the potential to provide a sound scientific basis for the justification of values for trigger levels.

TARPs often contain “and” and “or” statements. “Or” statements are values that can trigger an increase in the TARP level on their own. “And” statements require two or more values to be achieved before an increase in the TARP level. The decision of whether to use one or the other, or even a combination of both within a TARP is dependent on many factors and requires a sound understanding of spontaneous combustion. For example, hydrogen is an indicator of spontaneous combustion but elevated hydrogen levels are also known to be associated with the use of galvanised steel. The use of an “or” statement for hydrogen in a TARP could result in an unnecessary elevation of the TARP level due to the use of galvanised steel, rather than an actual heating. A solution to this problem would be the use of an “and” statement, so that elevated hydrogen occurring must be in conjunction with another elevated spontaneous combustion indicator (carbon monoxide for example), to result in an increase in the TARP level.

Consideration must also be given to how the data used in a TARP is generated. The gas chromatograph (GC) is the only available technique for the measurement of the spontaneous combustion indicator ethylene. Most modern GCs are capable of low level ethylene detection from approximately 1ppm and upwards. There are difficulties in interpreting whether ethylene is present below 1ppm due to the nature of the technology. The use of an “or” statement for the presence of ethylene as an evacuation trigger is not advisable due to this. Ethylene typically develops at higher temperatures in coal and therefore its presence due to a heating will be in conjunction with the presence of other elevated indicators. A more
appropriate use of ethylene in a TARP would be with an “And” statement along with another spontaneous combustion indicator.

The oxygen concentration at a seal is an example of the appropriate use of an OR statement. Oxygen ingress at seals can lead to spontaneous combustion, therefore the need for an AND statement with another qualifier would not be required before taking action. The presence of oxygen itself is enough to trigger the need for immediate action.
Figure 3: gas data from 512 Seal Moura No 2 1994

Figure 4: Segas Professional Coward Triangle
CONCLUSIONS

This paper discusses the main requirements that need to be considered when installing tube bundle underground monitoring systems at a coal mine. The risk of electrical equipment being underground is removed by using the tube bundle system, as its analysing and controlling hardware is established on the surface.

The main points to consider are tube bundle installation, maintenance and optimisation requirements, as well as training requirements for both the system maintenance and system software operation. The main goal of monitoring the underground atmosphere is to determine the gas mixture explosion risk, the early identification of the onset of a spontaneous combustion event and the verification of the effectiveness of inertisation techniques. The system is also used to make the decision to re-enter or not re-enter the mine after an emergency evacuation.

The climate is an important factor to consider when installing a tube bundle system. The tubes are food grade polyethylene and as such are prone to weather damage. The insulation and heating in cold climate and UV protection (for example running them in pipes) is recommended. It is recommended that 12.7 mm (0.5 inch) outer diameter (OD) tube is used for runs up to 6 km and thereafter 15.9 mm (5/8 inch) OD tubing is used.

Where practicable, individual tube flow regulators should be used on all tubes to equalise the flow in each of the tubes. Good quality tubing and joiners, as well as technical capabilities of personnel installing the system are vital for maintaining safe operation and to avoid leaks during installation and operation. The correct leak testing procedures need to be established and performed monthly on all tubes and after a relocation or new installation. During the installation stage, possible condensation points need to be considered and where possible tubes should be run in the return or conveyor belt roadways. Regular checks of tube monitoring locations is very important to ensure correct calculation of gas values and correct mine atmosphere status.
The tube bundle monitoring software is a very important part of the system. It has functionality for settings for different operational access levels and permissions to acknowledge alarms, trend the data, hold the sampling of the tube, calibrate and troubleshoot the analyser, or higher permission to change configuration and set the alarms. Individual alarms need to be stabilised for each of the tubes for all the gases being monitored (CO, CO₂, CH₄ and O₂) as well as the relevant ratios Grahams, CO/CO₂, Explosibility and CO make. The calibration of the analyser and integrity testing is an essential part of the operation and all operators need to have a high level of understanding of the processes required. All essential spare parts should be stored on site and be readily available in case of equipment failure.

Queensland or New South Wales Mining Regulations need to be considered when setting the alarms and ratio calculations within the tube bundle operating software. The alarm number and threshold needs to be justifiable and statistical analysis can be used to establish the appropriate alarm levels (Stephan 2000). TARPs are used to define what are normal and abnormal underground atmosphere conditions, and the appropriate actions that should be undertaken should certain levels be triggered.

Tube bundle gas analysis information can be displayed in time graphs, trends, explosibility triangles and diagrams (Coward Triangle and Ellicott Diagram).

**RECOMMENDATIONS**

1. Tube bundle should be run in mine roadways where condensation will be minimised and secured to the roof at a minimum of 2 m intervals to improve the survivability in the event there is an explosion in the mine.
2. Individual line regulation be used to balance the flows in the tubes attached to each purge pump
3. Coal mines should establish a working relationship with the supplier of their tube bundle hardware and software to ensure regular maintenance and training is undertaken.
4. Critical spares should be held on site.
5. Systems to be maintained in accordance with AS2290 Part 3
6. The gas data is backed up to a secure storage server to prevent data loss from failed hard drives, lightning damage, server theft.
7. Trigger Action Response Plans (TARPs) need to be established carefully considering many factors, such as coal seam and history, laboratory testing conditions, tube bundle system conditions and mining methods.
8. Care should be used in setting TARP levels and the appropriate use of “and” and “or” statements needs consideration and evaluation for appropriateness.
9. TARPS should be clear, unambiguous and able to be clearly understood by a coal mine worker in the early hours of the morning, when no technical staff are available.
10. Regular mine emergency exercises should be performed with coal mine tube bundle simulation software.

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OHS in the Mining Industry in the 21st Century

David Cliff

Abstract: As the 21st century progresses serious questions are being asked of the capacity of the minerals industry to maintain or improve its Occupational Health and Safety (OHS) performance. The pressures on mining and mineral processing will come from many directions. Cost pressures are only going to increase; societal acceptance of mining may well become harder to obtain and mining and mineral processing conditions will probably get worse as lower quality ore bodies have to be mined at deeper depths. This in turn places pressure on OHS; in the developed world there will be smaller work forces operating more autonomous machines. Fewer people, lower exposure and therefore less risk? Not necessarily, as the danger of low manning levels is that the knowledge and awareness of the risks also diminishes. As incidents become less frequent, the awareness of them and their potential for harm reduces, actually increasing the risk of harm. In the developing world, due to low labour costs, ongoing unemployment issues and in many cases less rigorous legislative requirements, there will undoubtedly be an increase in small scale and artisanal mining, as well as the development of large scale mines. Here the potential for harm is due to the total lack of awareness of OHS issues at all levels in the industry and the lack of regulatory capacity to promote and enforce a safe and healthy workplace.

INTRODUCTION

Within the developed world great progress has been made in reducing accidents and incidents in the mining industry. Large highly mechanised mines, employing relatively small workforces, have achieved significant reductions in frequency rates. Australian mines are a good example of this. Figure 1 shows the Fatal Injury Frequency Rate (FIFR) (deaths per million working hours) since 1989. For convert to per 1000 workers simply multiply by 2, as the average Australian miner works about 2000 hours per year. Going back further would show even higher fatality rates.

Figure 1: Fatal Injury Frequency Rate for Australian Mining since 1989

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The risks posed vary with the type of mining. Figure 2 depicts a comparison between the major mining sectors over a ten year period for a number of countries. In addition the risks have changed over time. Historically principal hazards such as fire, explosion, outburst, rockburst and fall of ground have been the major cause of fatalities. These hazards are generally well managed today and the focus has shifted to incidents that cause individual rather than multiple fatalities as shown in Figure 3.

![10 year FIFR (per million working hours)](image)

**Figure 2: 10 year FIFR by sector**

Fatalities in Australian coal mining as a result of Principal and Non-principal hazards for two consecutive decades (1991-2000 & 2001-2010)

![Diagram](image)

**Figure 3: The change in the nature of fatalities in Australian coal mines over the past 20 years**

(Kirsch et al., 2014)

*Note: N\(^1\) = total number of fatalities between 1991 and 2000; N\(^2\) = total number of fatalities between 2001 and 2010. OC = Open Cut; UG = Underground. *one fatality that occurred between 2001 to 2010 is not included in the total as it could not be sorted according to the listed hazards.*
In Australian mines, most fatalities over the past 10 years, with the exception of rock falls, have involved plant or equipment in some way (figure 4).

**Figure 4: The number and type of fatal accidents in Australian mines over the past 10 years**

The fatality curve shown in Figure 1 is typically aligned with changes in safety culture as shown in Figure 5 and the reduction in rates is due to the progressive implementation of technology, systems and culture (or a people focus) as shown in Figure 6. Technology continues to develop with the introduction of autonomous vehicles and the widespread use of remote control equipment.

**Figure 5: The Bradley Curve (Dupont 2015)**
There is no doubt that through the implementation of technology and through gaining an understanding of major mining hazards, the incidence of multiple fatalities has been reduced. The last fire or explosion in an underground coal mine that caused fatalities was in 1994. Australia went over 10 years without a multiple fatality event (2000 to 2013).

The implementation of safety management systems, risk management, work force representation and duty of care and process based legislation have been credited with making Australian mining the safest in the world.

Sophisticated incident investigation techniques that allow investigators to drill down to the underlying causes have assisted in developing safe systems of work. These underlying causes do not always rely on an error by the person who was harmed. The importance of external factors such as the organisational culture, the work environment, the physical environment and the actions or inactions of others have now been recognised.

Huge strides have been made in improving health and safety performance by shifting from compliance with legislation to management with best practice standards.

The current low commodity prices place immense pressure on mines to improve productivity and reduce staffing. This in turn can lead to a reversal of the safety culture improvement through the focus on doing what has to be done rather than what should be done. Brown et al., (2000), while exploring safe employee behaviour in the steel industry, found that during times of increased production, employees felt that the need to meet production quotas abated safety procedures, and that their bonuses and jobs may be placed in jeopardy if they were to follow these procedures.

Technology, systems and culture all require continual effort and adequate resourcing to be effective. Reducing the effort or resourcing risks increasing the health and safety risk. Success is its own worst enemy. The low fatality rate breeds complacency, and the assumption that the hazards are not real as miners have not experienced them themselves. This in turn leads to the underestimating of risk and therefore reducing the need for controls. In the past two years in Australia there have been a spate of fatalities and the rate is much higher than for the previous four years, this may just be coincidence or a warning of things to come. The Pike River Coal Mine disaster where 29 miners lost their lives in 2010 (Royal Commission 2012), is seen as a classic example of failing to recognise the hazards. With limited
resources there is a tendency to retreat to rules and compliance, compliance of course aims to ensure a minimum standard not best practise. This would suggest a reversion in safety culture to earlier more basic levels. There is a need for companies to reassert their commitment to the health and safety of workers and ensure that no corners are cut.

Effective safety and health management systems require continual vigilance to ensure that the systems are being implemented as designed and are achieving the desired outcomes. This monitoring and review process requires personnel, resources and management commitment. The focus needs to be on the effectiveness of controls not just having controls. Sometimes there are too many controls and it can be better to focus on a few critical controls that are effectively implemented (Hassall et al., 2015).

New technology will be introduced to improve mining and mineral processing, but unless how it works is properly understood, there is the danger of creating new risks not from malfunctioning systems but from complex systems functioning as designed, just not as predicted (Dekker 2011). An example of this is the recent accident between an autonomous haul truck and a water cart. The autonomous haul truck performed as it was programmed, however the driver of the water cart was not aware that the vehicle was about to turn across its path (Latimer 2015). The “Human Error” was not by the driver of the water cart but more likely by the people responsible for the design and implementation of the autonomous trucks and the need to coexist with manually driven vehicles.

To avoid this we need to focus on the right hand side of the safety culture diagram shown in Figure 5 and recognise that the workforce is an asset that needs to be nurtured. Now more than ever the old mantra of a safe workforce is a productive workforce is true. Fiedler back in 1984, (Fiedler et al, 1984) demonstrated that working on productivity and safety cooperatively improved both. All those attributes listed under the Independent and Interdependent categories are not exclusive to OH and S. They apply equally to effective production. McLain and Jarrell investigated the outcomes of compatibility between safety and production. In line with theory, this research suggested that safety-production compatibility (and thus conflict) was linked to safe work behaviours and the extent to which hazards interfered with tasks performed (McLain and Jarrell 2007).

The financial benefits of good OHS performance are well documented. The costs include not only the direct costs but also the hidden costs which can be up to 200 % of the direct costs (Oxenburgh 1991). Thus it would be argued that reducing accidents and illness makes good financial as well as ideological sense. It is not just time away from work that reduces productivity but also presenteeism – where one is at work but not functioning at full capacity, also reduces productivity. Williden et al., (2012) showed that workers affected by stress and anxiety whilst still at work, have reduced productivity by over 10 %. Presenteeism can multiply the real cost of an illness by up to four times (Geotzel et al., 2004).

As well as causing illness and lost productivity, work stress has also been shown to influence employee safety through a number of mechanisms. Masia and Pienaar (2011) found that work stress had an inverse relationship with safety compliance, as they did for job insecurity. In addition, several studies by Maiti and colleagues have suggested that job stress encourages employees to avoid safe work behaviours, thus increasing their likelihood of workplace injuries, and that job stress can indirectly lead to employees becoming less job-involved, which may also increase their likelihood of injury as greater job involvement is associated with better safety performance (Maiti 2004; Paul and Maiti 2008). In the same studies, Maiti and colleagues also suggested associations between safe work behaviours and negatively personified individuals, suggesting that these individuals not only fail to avoid work injuries, but that they are also unable to extend safe work behaviours in their work but instead engage in risk-taking behaviours, all of which makes them more susceptible to workplace injuries (Maiti et al., 2004; Paul and Maiti 2008).

The level of stress in professional staff must increase as their number decrease. There will be fewer of them, particularly in support roles, to cope with all necessary tasks and responsibility. This may lead to high turnover of key roles and loss of corporate memory. Time pressures may cause tasks to be done
quickly rather than completely. Risk assessments are in danger of becoming paper exercises rather than real assessments of risk.

These stressors are worse when we consider the other world of mining prevalent in developing countries – Small Scale and Artisanal Mining (SSAM). This type of mining is already characterised by generally poor technology, no systems and poor culture. It is mining to survive. It is difficult to know how many people die, are injured or suffer ill health each year in SSAM as no statistics are gathered for what is informal and often illegal mining. This kind of mining is likely to increase in scale as the world population increases and more and more people scramble to survive. Small-Scale and Artisanal Mining occurs in approximately 80 countries worldwide. The sheer scale and transient nature of the mining makes it difficult for governments to manage, especially those who are poor. Artisanal and small-scale production supply accounts for 80% of global sapphire, 20% of gold mining and up to 20% of diamond mining (World Bank 2013). It is widespread in developing countries in Africa, Asia, Oceania, and Central and South America. Though the informal nature and on the whole un-mechanized operation generally results in low productivity, the sector represents an important livelihood and income source for the poverty affected local population. It ensures the existence for millions of families in rural areas of developing countries. About 100 million people – workers and their families - depend on artisanal mining compared to about 7 million people worldwide in industrial mining (World Bank 2013). Landslides are common; the use of toxic chemicals, such as mercury, is widespread; and women and children are often pressed into service in the most menial and dangerous roles (Navch et al., 2006). Mining and mineral processing often occur in and around communities exposing not only the workers but their families to the hazards. Whilst there has been a concerted effort to remove mercury from gold processing in SSAM, the primary aim of the program is the reduction in environmental harm rather than the harm to the miners. There is a need for a concerted effort to improve the health and safety of SSAM miners through educating them in the risks and the controls, provision of support and equipment and encouragement to use safer techniques. It is also vital that this type of mining is recognised and legitimised. Governments must exercise a much greater role in facilitating the health and safety of artisanal miners. Wealthy countries need to support activities in these countries aimed at improving OHS and the quality of life.

CONCLUSIONS

The 21st century poses many challenges to improving the health and safety performance of the world-wide mining community.

In some ways the recent success in reducing the fatality rate in mining in developed countries is its own worst enemy as it can breed complacency. Couple this with the pressures to reduce cost and improve productivity and there is a real danger that the health and safety performance in developed countries will get worse rather than better. It is essential that we fully understand how new technology works and the full implications of its operation.

For the developing world the situation is potentially grimmer with increased pressure from a rapidly growing population merely seeking to survive, not able to afford the niceties of safety equipment or good work practices.

Both worlds will be impacted by low commodity prices leaving less money to spend on discretionary items. The challenge is to not make good health and safety optional but compulsory a normal part of the way of life.

REFERENCES


ABSTRACT: This paper will present current investigations into the use of vision enhancement and scanning technology that can enable the navigation and manoeuvrability of a self-escape vehicle through dense dust and smoke. This paper will also present research into instrumentation and communication for obtaining information following an emergency event.

INTRODUCTION

In the aftermath of recent major mine disasters there has been much debate around tactical and emergency response and preparedness. This falls into two broad categories: (1) self-escape, and (2) rescue and recovery. In these situations, the primary objective is always to facilitate and enable the self-escape of mine personnel to safety in the first instance. However, this is often not possible and crucial time-critical decisions are required of the emergency services. Common shortfalls in emergency response includes the failure to capture and/or process key information, lack of mine environmental information (current and/or previous information leading up to such an event), limited knowledge of workforce status and location, and consequently increased uncertainty in determining the risk for the response and recovery team tasked with re-entering the mine.

Simtars are currently engaged in research into developing and testing optical and scanning technology that can be retrofitted to a mine vehicle to provide navigation assistance and thus enable escape during emergency events. One of the key issues is that loss of visual sight and location awareness, particularly due to the presence of smoke and dust, can significantly impair or even completely prevent the ability for a mine worker to self-navigate the vehicle out of a mine even in low concentrations (Kissell and Litton 1992). Therefore, technology is needed to aid the driver’s own ability to locate their relative position, direction of travel and avoid impacting obstacles and/or the mine walls.

Previous tests were carried out at Simtars in 2013 using the SICK laser scanning (LIDAR-based) technology. A comprehensive set of trials were conducted to test the SICK laser scanners’ performance under outdoor conditions, and in the presence of both dust and smoke. Tests were also carried out using a polycarbonate, or ‘Lexan’, clear screen with the view that this would be required in an Ex-certified end product. The main results demonstrated the severe limitations that both smoke and dust cause. In most cases the absolute maximum range achieved under worst-case conditions was 5 m, and in some instances the target obstacle was only detected at a maximum of 2 m distance. Due to the nature and conditions of these previous trials, the smoke tests obtained were qualitative due to the environment used, and lack of equipment available to measure the smoke quantity at the time. However, different types of smoke were trialled: ‘Black smoke’ through burning a mixture of fuels, and other substances to generate a think soot-rich smoke, and ‘white smoke’ through wood and other materials. As expected, the black smoke had a significantly increased adverse effect on the measurements opposed to white smoke. Whist the qualitative data was limited these trials provided a useful reference indication of performance for future work. Testing was also carried out using coal dust in ‘worst case’ concentrations of ~60 g/m$^3$. The effect of the dust on the laser scanning equipment was more severe than the smoke test results.

During the recent testing programme, Simtars have been conducting a series of tests in different environments ranging from small-scale through to large scale live fire tests. Simtars have recently constructed a purpose built Dust and Smoke Test Chamber with instrumentation to quantify the

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measurements. All testing, including the results presented in this paper were carried out using this test facility, except for the live fire tests that were carried out at Queensland Fire and Emergency Services (QFES) Live Fire Training Centre.

The vision enhancement technologies currently being trialled by Simtars are Short Wave Infrared (SWIR) and Long Wave Infrared (LWIR). SWIR technology, typically used across the 900 nm (nanometers) to 1700 nm wavelengths, uses Indium Gallium Arsenide (InGaAs) sensors for detection. It is beyond the visible spectrum and is often used at 1550 nm, which is eye-safe.

LWIR, or more commonly referred to as thermal-infrared, technology is typically used in the 8 - 12 µm wavelength range. It is a highly established technology for heat measurement among many other thermographic applications. In theory, the longer wavelength will have better penetration through small particle clouds, however the images are completely restricted to heat sources and it is likely that the images will become quickly saturated in smoke and dust. Figure 1 shows the visible, SWIR and LWIR spectrums.

![Figure 1: Diagram showing the visible, SWIR and LWIR spectrums (from UTC Aerospace Systems 2016)](image1.png)

TEST EQUIPMENT

Simtars has built a dust and smoke test chamber at its headquarters in Redbank. It consists of three 40’ length shipping containers joined end to end, covered externally with 100 mm thick insulation panelling. Figure 2 shows the exterior of the chamber and the exhaust fan.

![Figure 2: Simtars Dust and Smoke Test Chamber, the rear of the test chamber is shown with the exhaust fan](image2.png)
The exhaust fan at the rear of the chamber has a diameter of 630 mm. It has a variable speed controller. This allows the fan to be turned on such that it applies only a small pressure differential to the test chamber while a test is being conducted. This prevents dust and smoke from entering the control room. When a test is finished, the fan speed can be increased to quickly evacuate the test chamber of dust and smoke.

Internally the containers have been divided into three rooms: a monitoring and control room of approximately 12 m in length, a test room approximately 12 m in length and an exhausting chamber of approximately 12 m in length. The control room allows the operators to remotely fill the test room with smoke and monitor the data being recorded, such as the opacity of the atmosphere.

A Sick DustHunter is used to monitor the opacity of the test room. This unit was chosen as in order to provide continuous reliable results it uses an air purging system to prevent any clogging of dust around the sensors. Barrel fans are placed on the floor of the test room to mix the dust/smoke atmosphere and assist in achieving a homogenous mixture.

As shown in Figure 3, a “target” frame is mounted on a pulley system so that its position can be moved, from control room, during a test. The frame is designed to accommodate difference payloads such as light targets or objects. A wire line encoder has also been installed to accurately record the distance, or longitudinal position, of the frame during testing. The LED light beacons, as shown in Figure 3, can also be installed on the target frame. The red, blue, green and amber colours are used for the different wavelengths across the visible spectrum. They also give a visual reference point from the control room as to how opaque is the atmosphere in the test room. Testing conducted by NIOSH suggests that in underground mine environments green light is more easily detected through smoke in emergency conditions due to the typical low-light environment, presence of dust, and the typically high average age of the workforce (Sammarco 2012). As part of this study Simtars are also evaluating the visibility of the beacons in both smoke and dust conditions.

![Figure 3: The test room inside the Simtars Dust and Smoke Test Chamber, note that the moveable target frame with the LED beacons, also visible are fans for circulating dust and the opacity meter](image-url)
In addition to the LED light beacons in the visible spectrum, LED SWIR beacons are used to test the ability of the SWIR camera to see through dust and smoke. They emit at either 1050 nm or 1550 nm. They are shown in Figure 4. The LED SWIR beacons were specifically made for this study by the manufacturer.

![LED SWIR beacons](image)

**Figure 4**: LED SWIR 1050 nm and 1550 nm beacons and a SWIR Lambertian Reflector

Similar to the above application described for the SWIR beacons, Simtars have also investigated the use of thermal IR beacons for detection through the presence of dust. Figure 5 shows the thermal IR beacon that will be used. It emits in the 8-12 micron spectrum.

![Thermal IR beacon](image)

**Figure 5**: MS-OMR II (8-12 model) Beacon (from Thermal Beacon 2003)

Simtars is currently evaluating a SWIR camera and two thermal infra-red cameras. The images from these cameras are compared to footage taken with a standard CCTV camera. Table 1 lists the model details and image type of the cameras that have been tested by Simtars to date. Other technologies are also being considered for testing in the Simtars Dust and Smoke Test Chamber, for example Lidar systems, Radar, and other wireless navigation systems.

<table>
<thead>
<tr>
<th>Camera Model</th>
<th>Image Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Photonics Science SWIR</td>
<td>Short wave infra-red (SWIR)</td>
</tr>
<tr>
<td>FLIR PathFindIR</td>
<td>Long wave infra-red (LWIR)</td>
</tr>
<tr>
<td>Nautitech Thermal Infra-Red</td>
<td>Long wave infra-red (LWIR)</td>
</tr>
<tr>
<td>Sony CCD Monochrome</td>
<td>Visible CCD</td>
</tr>
</tbody>
</table>

**Table 1**: Cameras used in low visibility tests conducted by Simtars to date

The Queensland Fire and Emergency Service (QFES) facility on Whyte Island has the capability of conducting live fire tests. In conjunction with QFES, and University of Queensland (UQ) Simtars conducted a series of tests at this facility using a standard CCTV camera, SWIR camera, and two thermal LWIR cameras. The test building that was used consisted of two levels, with multiple rooms on
each level. A fire was initiated in an adjacent room (burn room) and the smoke from this fire was allowed to build and pass into a second room (smoke testing room), in which light beacons and an opacity meter were set up. In a third room (camera room), cameras were positioned such that the light beacons were able to be monitored. Figure 6 shows the layout of the rooms used. The distance of the smoke testing room is 10 m.

The fuel used in all fire experiments was kerosene. The density (and hence soot-content) was controlled by limiting the ventilation into the burn area. The smoke was allowed to build through the stages of the fire into the smoke testing room. The smoke opacity meter was used to record the smoke levels in real-time throughout the testing. LED beacons were also used to record the visual visibility by the human eye throughout the testing. Figure 7 shows photos of the LEDs in the smoke test room during different progressing stages of a fire test.

![Figure 6: Layout of rooms used for testing various camera technologies at QFES Whyte Island facility](image1)

![Figure 7: Photos taken during Live Fire Tests at different stages of test](image2)

Figure 8 shows the images taken at the start of a test conducted before smoke was introduced into the room. The top left image shows the image recorded with a standard CCTV camera. The top right image shows the image taken with a FLIR thermal LWIR camera, the bottom right image was taken with an IECEx thermal LWIR camera. The bottom left image was taken with a Photonics Science SWIR camera. Note that field of vision for each of the cameras is the same. The widest field is shown with the CCTV camera. A doorway is clearly seen in this image, and it is at or through this doorway that the other cameras are focused. A useful reference point is the railing that can be seen at the centre bottom of the CCTV image, and which can be seen in the other three images.
Smoke was introduced to the room until 90% opacity was reached. For comparison, it is no longer possible for a person to see their hand in front of their face when 30% opacity is reached. Figure 9 shows what can be seen by the various cameras at this point in time.

As is to be expected, the CCTV image is completely obscured. The thermal LWIR cameras show a clear image due to the increasing contrast in heat being emitted from the various objects in the room. The Photonic Science SWIR image is also good, though the image quality may be slightly degraded.

As the room cools down, the thermal IR images degrade. Figure 10 shows the images taken on another test. Smoke had previously been introduced to the room and ventilation has been applied to the room to remove the smoke. The top images are those taken with the CCTV camera and the bottom images were
taken with the FLIR thermal IR camera. The left images were taken just after the smoke was cleared. The right images were taken 20 minutes later.

Figure 10: Top images are taken by a standard CCTV camera, the bottom images by a Thermal LWIR camera, the images on the left side show the room after a smoke test and the contents of the room are still warm, the images on the right show the room after it has cooled down for 20 min and the degradation in image quality of the Thermal IR camera can be seen.

There is a marginal increase in image quality of the CCTV camera as the remnants of the smoke are cleared. The thermal LWIR image, however, decreases significantly in quality. This is due to the room cooling and the temperature difference between the various objects in the room decreasing.

In an underground mine situation, it is not expected that there will be a significant temperature gradient between various objects unless the miners are next to a fire. As such, the thermal IR cameras are not the optimum camera for use on a mine escape vehicle. The SWIR camera images were good across a wide variety of visibility and temperature conditions. A new SWIR camera is in the process of being commissioned at Simtars and further testing will be conducted using this type of camera as the initial results observed are very promising.

CONCLUSIONS

This paper has presented the scope of work that Simtars is currently undertaking into optical and scanning technology for aiding mine escape vehicle navigation. A summary of the initial results from the live fire testing has been presented, with IR camera technology showing very promising results to date. Further testing will be carried out using other technologies including a new un-cooled SWIR camera, LiDAR based scanning and other radio based navigation technologies. Figure 11 shows an image of Time of Flight (ToF) based wireless sensor network technology that Simtars is currently evaluating for positioning and navigation technologies.
A series of further tests will be carried in 2016 using the bespoke Smoke and Dust Chamber to identify and refine the appropriate technology. Following on from this an underground mine trial using these technologies will be conducted.

Figure 11: Wireless sensor network based positioning system

Related to this research area, Simtars is currently engaged in developing technology for emergency response applications. This will rely on adopting a resilient wireless communication system that will be initiated following an emergency event in order to collect and transmit vital information to the surface. The technology functionality and performance of the wireless sensor networks is largely understood in underground mining, however, a key requirement of such a system will be the survivability in any explosion or rock fall. Such technology may be of benefit for both aiding the self-escape of mine personnel and also in providing vital information to aid in key decision-making at the surface during an emergency event.

ACKNOWLEDGEMENTS

Simtars wish to acknowledge the Mine Escape Vehicle Committee for their ongoing support, and also the Queensland Fire and Emergency Service, University of Queensland’s Fire Research Group, Nautitech Mining Systems, CSIRO and SICK for their collaboration and contribution to this work.

REFERENCES


A SYSTEMATIC APPROACH TO EVALUATE THE ROLE OF VIRTUAL REALITY AS A SAFETY TRAINING TOOL IN THE CONTEXT OF THE MINING INDUSTRY

Shiva Pedram¹, Pascal Perez¹, Stephen Palmisano² and Matthew Farrelly³

ABSTRACT: The Australian mining industry has achieved impressive performance and safety results through continuous improvement of its training standards. Interactive virtual reality-based training is the most recent technology used to enhance workers’ competencies in a safe and controlled environment which allows the replicable testing of extreme event scenarios. Like any other training method, Virtual reality (VR) -based training must be assessed in order to evaluate the advantages and limitations of this innovative technology, compared with more traditional approaches. Research was aimed at designing and implementing a framework to tackle the cultural issues involved in accepting innovative VR-based training programs developed for high risk industries. The present study was conducted with Coal Services Pty Ltd, a pioneering training provider for the coal mining industry in NSW, Australia. The research focussed on specific training programs developed for the mine rescue brigades. These brigade teams are made up of highly specialized miner volunteers who provide the primary response to major incidents. The research framework examined the adequacy of training needs, technological capabilities and the implementation of interactive simulation. The research outcomes provide evidence-based information on the advantages and limitations of VR-based training for mining rescue brigades. The framework is flexible and can be applied to other types of training for the mining industry or adapted for use in other industries.

INTRODUCTION

Computer simulation as a learning environment has progressively embraced technological innovations ranging from chart-based interfaces to fully immersive environments. (Bell et al., 1990 and Jou and Wang 2012). Virtual Reality (VR) provides both immersive and interactive features, allowing users to ‘feel’ that they are actually in the training environment (Raskind et al., 2005). Best practice in the mining industry includes extensive initial and professional training for staff involved in field operations. Simulator-based training is now frequently used to both establish and maintain this training. A VR environment, which is an interactive 3-D representation of the mine, has a high potential to enhance miners’ safety through improved techniques for training, retraining and up-skilling.

During an emergency, rescue brigades are the first teams responding to a mining incident. Their members are highly skilled volunteers, selected by mine managers at each production pit. Rescue brigades attend frequent training sessions in order to perform effectively in an emergency situation. A VR-based training program for rescue brigades provides a safe environment to perform collective drills for various emergency scenarios. During these sessions, trainees can improve their technical and non-technical skills. Previous research has shown that flight simulators are very successful at bringing learning and theory into practice in a supervised, safe but highly realistic environment (Deaton et al., 2005). Despite the rapid development of VR-based training in the mining industry there has been little (if any) formal evaluation of its impact on miner’s skills and competencies. Furthermore, due to the specificity of underground mining, it would be dangerously misleading to extrapolate training transfer results from other industries such as aeronautics and automotive.

¹ SMART Infrastructure Facility, University of Wollongong, Email: spedram@uow.edu.au, Tel: 0432509790, ² Associate Professor, Faculty of Social science, University of Wollongong, ³ Coal Services, Woonona, NSW, 2517, Email: matthew.farrelly@coalservices.com.au,
Hence, we describe in this paper an experimental design aimed at introducing a systematic framework to better understand and evaluate VR-based training programs. In this study we conducted the research on VR-based training developed by Coal Services Ltd for underground rescue brigades in NSW, Australia.

Why do Accidents Happen?

Historically the mining industry is one of the most hazardous industrial sectors. Although the industry has achieved significant success in reducing the number of accidents and limiting their consequences, it remains a risky business. According to NSW Trade and Investment, the average Fatal Injury Frequency Rate (FIFR) decreased by 65% between 2007 and 2012. The overall Lost Time Injury Frequency Rate (LTIFR) also decreased by 58% over the same period while the serious bodily injury frequency rate (SBIFR) decreased by 56% (Trade and Investment Resources and Energy, 2013). These records suggest that the Australian mining industry has achieved remarkable improvements through continuous development of its safety procedures. The bulk of the remaining accidents appear to be due to human error, as has been shown by Williamson (1990) in Australia and in the US. Sources of human errors are diverse and need to be integrated into relevant training programs.

The ‘Human Factor Analysis and Classification System’ (HFACS) is a systematic and evidence-based framework aimed to design, assess and enhance the interaction between individuals, technologies (including equipment) and the organisation (Wiegmann and Shappell, 2001). HFACS describes human error at each of four levels of failure: 1) unsafe acts of operators, 2) preconditions for unsafe acts, 3) unsafe supervision, and 4) organizational influences and outside factors (Patterson and Shappell 2010). HFACS has been implemented in various hazardous industries such as civil aviation (Wiegmann and Shappell 2001; Wiegmann et al., 2005 and Shappell et al., 2007), air traffic control (Broach and Dollar 2002), logistics (Reinach and Viale 2006 and Baysari et al., 2008 and Celik and Cebi 2009), and medicine (El Bardissi et al., 2007).

According to the HFACS framework (Figure 1), human errors should be minimised if appropriate training programs have been put in to the place.

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**Figure 1: Human Error Classification (Trade and Investment 2013)**
VR-based Training for Rescue Brigades

Misanchuk (1984) lists the three main factors used to evaluate the quality of a training session or program: (1) the employee’s ability to accomplish the assigned task, (2) the relevance of the training materials to what trainees are expected to do, and (3) the employee’s motivation to undertake training. Oftentimes, this technique of evaluation works however the case is different if we are aiming at measuring safety outcomes. Therefore, we cannot measure and evaluate the success of training through its impact directly on the quality of workers. Figure 2 shows how Coal Services Pty Ltd is developing the content for VR training sessions and how they are aiming to measure the training outcomes through the annual underground coal mine competitions.

Training requirements and guidelines are based on Australian Coal Mine Health and Safety Acts and regulations. Hence, training transfer can be evaluated against the following team performance, team effectiveness, pre-use equipment checks, fresh-air base, trauma management, use of the compressed air breathing apparatus and fire-fighting skills.

However, it must be mentioned that the rescue brigades undergo through six rounds of different training each year. As only one of these training modules is conducted in VR, it is not straightforward to determine the actual contribution of VR training to performance. Currently ambiguity exists as to whether VR is actually a successful training tool and if it is responsible for significant changes to the expected safety training outcomes.

In the next section a systematic approach is proposed which should enable industry to investigate and clarify the role of VR as a training tool (i.e. to determine whether it will fulfill the training requirements and identify its shortcomings). This approach: (1) identifies the specific training needs, (2) identifies any issues that will be faced if they choose the “real life training approach” and (3) informs them what the VR’s capabilities are and what are users’ opinion on its identified capabilities.

Participants

This study includes 280 Brigades men who were experienced underground miners and voluntarily joined the mine rescue brigades. We chose this group of miners since we could do a follow up study on them.
and monitor their performance in real life. With the other groups the chances were low that we would be able to monitor their performance in future: e.g. there are unemployed miners who attend the induction training courses at Coal Services to be prepared for future employment, since there is no guarantee when they are going to be employed, they were excluded from this study and only focussed on rescue brigades who are currently employed and in case of emergencies will be deployed. Moreover, VR development team and trainers (10 trainers) have been interviewed.

**FRAMEWORK**

The framework (Figure 3) includes four nested layers of analysis. Gaps and mismatches at the interface between two layers will help to identify training deficiencies and possible improvements to the current programs set up by the industry.

![](image)

**Figure 3: Evaluation framework**

The outermost layer of the framework corresponds to actual training needs. Interviews with trainers, mine managers and station managers constitute the main source of information alongside reviews of the literature produced by the mining industry. The second layer focuses on constraints associated with real-world training (aka traditional training). The third layer focusses on capabilities associated with VR technology. In-depth interviews with VR designers will help to better understand potential and actual use of this technology. Finally, the innermost layer corresponds to the learning process experienced by trainees. Over a two-year period, several rescue brigades have been followed through their training programs, focussing on VR-based training sessions.

**Actual training needs**

Need analysis is required to first identify the users’ needs, and then assess the need to recognize the importance and relevance of the identified problem and solution. Need is defined as a problem of the target group which can be solved (McKillip 1987). Based on McKillip (1987) there are five main steps in need analysis: identifying the users and uses, describing target population, need identification, need assessment and finally communicating the results to the decision maker and other relevant stakeholders.

Training Need analysis starts with two questions (1) is the training tool adequate or not? And (2) if it is inadequate what can correct it? Subject Matter Experts (SMEs), such as trainers and mine managers, were interviewed to identify potential training needs and how those needs could be fulfilled (Figure 4).

**Real-world’s training constraints**

Focus groups, made up of SMEs, trainees and trainers were asked to identify: (1) the constraints associated with real-world training, and (2) the potential for VR-based training to overcome these limitations.

**VR-based training capabilities**
Based on the above, an initial set of desirable VR-based training features can be identified. Interviews with VR designers and trainers identified: (1) the current capabilities of VRs, (2) the limitations of VR and potential for upgrade, and (3) the relevance of VR technology features for training purposes. The objective was to identify: (1) the role of simulation features and resources in overcoming the identified training challenges, and (2) the challenges of using each simulation feature. The VR-based training environment used by coal services Ltd corresponds to a state-of-the-art 360° interactive theatre with 3-D immersive visualization.

**Figure 4: Need Analysis Framework**

**VR-based training utilisation**

Over a two-year period, several rescue brigades were followed through their training programs across four training facilities (Wollongong, Newcastle, Singleton and Lithgow in NSW). Each trainee had to complete a short questionnaire before and after each training session in order to record previous experiences, expectations, responses to VR environments and self-assessment of individual performance. Equivalent questionnaires were provided to the trainers as well.

**RESULTS**

**Actual training needs from trainees point of view**

It is crucial to identify what are the characteristics of a successful training tool/environment from different points of views in order to be able to compare it to alternative tools/environments (to determine what features are missing or need to be added or modified).

<table>
<thead>
<tr>
<th>Training Needs from Trainees Point of View</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Recreate the Real Conditions (such as smell, noise, temperature, dusk)</td>
</tr>
<tr>
<td>2. Physical Activities can be done</td>
</tr>
<tr>
<td>3. Accessible at any time training is needed</td>
</tr>
<tr>
<td>4. Faithfully recreate various real life scenarios</td>
</tr>
<tr>
<td>5. All the mines can be seen and experienced</td>
</tr>
<tr>
<td>6. Experiencing the hazard and danger</td>
</tr>
<tr>
<td>7. Minimum of distraction to the training process</td>
</tr>
<tr>
<td>8. Safe training environment</td>
</tr>
</tbody>
</table>

**Real-world’s training constraints**

Trainees were asked to identify the constraints they thought were associated with conducting training at actual mine sites. They indicated that training in the pit felt more realistic, however, they mentioned that there were some challenges which would affect training and ultimately learning outcomes. The table below summarises the reported constraints of real-world training:
Table 2: Real-World’s Training Constraints from Trainees point of view

<table>
<thead>
<tr>
<th>Real-World’s Training Constraints from Trainees point of view</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Pit training is realistic and physically too active</td>
</tr>
<tr>
<td>2. Pit training requires access and consent from mine operators</td>
</tr>
<tr>
<td>3. Pit training has logistical issues and time constraints</td>
</tr>
<tr>
<td>4. Pit training has less variety in scenarios/content</td>
</tr>
<tr>
<td>5. Pit training is not safe (it is higher risk, potentially hazardous)</td>
</tr>
<tr>
<td>6. Pit training has less review and discussion of the training session</td>
</tr>
<tr>
<td>7. Pit training engages actual resources</td>
</tr>
<tr>
<td>8. Combination (two or more of 1-7)</td>
</tr>
</tbody>
</table>

VR-based training capabilities from the VR-developers point of views:

Table 3 summarises the VR training capabilities identified by interviewing VR-developers. The VR capability list is long but here is provided a shortlist of the most relevant capabilities to this study:

Table 3: VR training capabilities from VR-developers point of view

<table>
<thead>
<tr>
<th>VR training Capabilities from VR-Developers point of view</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Powerful training tool when used correctly</td>
</tr>
<tr>
<td>2. Allows safe training on high-risk activities</td>
</tr>
<tr>
<td>3. Consultation between SME, RTO, industry and customer ensures quality training content</td>
</tr>
<tr>
<td>4. Done properly, simulation will complement an already existing quality training program</td>
</tr>
<tr>
<td>5. Simulation allows an additional form of training that can catch anything that may be missed by traditional methods</td>
</tr>
<tr>
<td>6. Allows regular refresher training in a time and cost effective manner</td>
</tr>
<tr>
<td>7. Use an agile development method to be flexible and deliver on a guaranteed shift in customer demands</td>
</tr>
<tr>
<td>8. Development includes collaboration with training authorities ensuring that training meets standards</td>
</tr>
<tr>
<td>9. By using blended learning, you ensure that all trainees get an opportunity to learn based on their own skill level</td>
</tr>
<tr>
<td>10. Can replace chunks of classroom learning and compliment practical training</td>
</tr>
<tr>
<td>11. Saves time and money while providing a wider variety of training scenarios</td>
</tr>
<tr>
<td>12. Will create better trained crew who have been exposed to a wider variety of training systems</td>
</tr>
<tr>
<td>13. Opportunity to get into simulation on the ground floor and get experience in best practice</td>
</tr>
<tr>
<td>14. If developed in a flexible manner, can allow customised training scenarios to cater to different trainees needs</td>
</tr>
<tr>
<td>15. To learn from any mistakes and make the business more productive</td>
</tr>
<tr>
<td>16. By introducing simulation as a compliment to traditional training, you minimise risk of intimidating resistant trainers/trainees.</td>
</tr>
</tbody>
</table>
VR-based training utilisation from various point of views

After trainees attended the training course in VR training environment they were asked to answer the following four questions:

1. What were the strengths of Virtual reality as a training environment?
2. What were the weaknesses of Virtual reality as a training environment?
3. What opportunities does Virtual reality provide as a training environment/tool?
4. What would prevent the use of Virtual reality as a training environment/tool?

In order to analyse the collected data we are using Strength, Weakness, Opportunity and Threat (SWOT) analysis. However, this technique does not reflect on knowledge creation and training transfer but provides a good insight about what position is VR holding at the moment and what is going to be the future of this kind of training environment. Therefore, for those who are willing to employ VR in a more systematic and meaningful way it could be of added value to inform the decision makers, strategy planners and training coordinators to what is possible to achieve with VR and what it holds for future.

This section presents a summary of different viewpoints gathered from the three main VR stakeholders who are, VR developers, trainers using VR as a training tool and trainees who are being trained in VR. Each user has their own concerns and requirements. In the following section a combination of their viewpoints are provided in Tables 4, 5 and 6.

**Table 4: SWOT from VR trainees point of view**

<table>
<thead>
<tr>
<th>Strengths</th>
<th>Weaknesses</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. VR provides a high level of fidelity and realism</td>
<td>1. VR produces Simulator Sickness</td>
</tr>
<tr>
<td>2. VR training is something different</td>
<td>2. VR does not fit the task</td>
</tr>
<tr>
<td>3. VR training allows real-time feedback and discussion</td>
<td>3. VR cannot replace real life training</td>
</tr>
<tr>
<td>4. VR allows training in a variety of different scenarios</td>
<td>4. VR does not allow me to be physically active</td>
</tr>
<tr>
<td>5. VR training avoids real world distractions</td>
<td>5. VR training is passive learning</td>
</tr>
<tr>
<td>6. VR training overcomes logistical constraints</td>
<td>6. VR training not run properly</td>
</tr>
<tr>
<td>7. VR allows safe training in high-risk activities (Controlled environment)</td>
<td>7. Combination (one or more of 1-6)</td>
</tr>
<tr>
<td>8. VR facilitates skill and competency creation/correction</td>
<td></td>
</tr>
<tr>
<td>9. VR technology is effective and easy to use</td>
<td></td>
</tr>
<tr>
<td>10. Combination (Two or more of 1-9)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Opportunities</th>
<th>Threats</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. VR can realistically simulate events and conditions (including dangerous ones)</td>
<td>1. Resistance to using the technology</td>
</tr>
<tr>
<td>2. VR training allows testing and maintenance of skill levels</td>
<td>2. Limitations of the technology</td>
</tr>
<tr>
<td>3. VR provides exposure to a variety of scenarios</td>
<td>3. Cost of the technology</td>
</tr>
<tr>
<td>4. VR training has better access and is more convenient</td>
<td>4. Simulator Sickness</td>
</tr>
<tr>
<td>5. VR provides more opportunity for discussion and feedback</td>
<td>5. Technical issues</td>
</tr>
<tr>
<td>6. VR provides a good introduction and initial experience</td>
<td>6. Training accessibility</td>
</tr>
<tr>
<td>7. VR technology facilitates training</td>
<td>7. Lack of good content</td>
</tr>
<tr>
<td>8. Suggestions</td>
<td>8. Not knowing how to use the technology</td>
</tr>
<tr>
<td></td>
<td>9. Combination (Two or more of 1-8)</td>
</tr>
</tbody>
</table>
### Table 5: SWOT from VR-Developers Point of View

<table>
<thead>
<tr>
<th>Strengths</th>
<th>Weaknesses</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Powerful training tool when used correctly</td>
<td>1. Expensive to start off</td>
</tr>
<tr>
<td>2. Allows safe training on high-risk activities</td>
<td>2. New methodologies and business practices need to be established</td>
</tr>
<tr>
<td>3. Consultation between SME, RTO, industry and customer ensures quality training content</td>
<td>3. Still requires practical training</td>
</tr>
<tr>
<td>4. Done properly, simulation will complement an already existing quality training program</td>
<td>4. Course creation is resource intensive</td>
</tr>
<tr>
<td>5. Simulation allows an additional form of training that can catch anything that may be missed by traditional methods</td>
<td>5. Requires development effort for best outcomes.</td>
</tr>
<tr>
<td>6. Allows regular refresher training in a time and cost effective manner</td>
<td>6. Off-the-shelf training packages may not deliver on all training requirements</td>
</tr>
<tr>
<td>7. Use an agile development method to be flexible and deliver on a guaranteed shift in customer demands</td>
<td>7. At this stage, technology doesn’t really allow major removal of traditional training methods</td>
</tr>
<tr>
<td>8. Development includes collaboration with training authorities ensuring that training meets standards</td>
<td>8. Difficult to prove improved training outcomes due to it being anecdotal in nature.</td>
</tr>
<tr>
<td>9. By using blended learning, you ensure that all trainees get an opportunity to learn based on their own skill level</td>
<td>9. Agile businesses are alien within the military/government space.</td>
</tr>
<tr>
<td>10.</td>
<td>10. Small minority may be resistant to change</td>
</tr>
<tr>
<td>11.</td>
<td>11. Seen as a game</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Opportunities</th>
<th>Threats</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Can replace chunks of classroom learning and compliment practical training</td>
<td>1. Seen as a luxury</td>
</tr>
<tr>
<td>2. Saves time and money while providing a wider variety of training scenarios</td>
<td>2. Being seen as a magic bullet, using it instead of practical training</td>
</tr>
<tr>
<td>3. Establish ownership by all parties</td>
<td>3. Preference to have agreement by all parties otherwise can be opened to criticism</td>
</tr>
<tr>
<td>4. Will create better trained crew who have been exposed to a wider variety of training systems</td>
<td>4. Expensive to initially develop a decent asset library</td>
</tr>
<tr>
<td>5. Opportunity to get into simulation on the ground floor and get experience in best practice</td>
<td>5. A small minority of the population can resist change which is a challenge that needs to be managed</td>
</tr>
<tr>
<td>6. If developed in a flexible manner, can allow customised training scenarios to cater to different trainees needs</td>
<td>6. If not done correctly may not deliver training outcomes that are expected</td>
</tr>
<tr>
<td>7. To learn from any mistakes and make the business more productive</td>
<td>7. Critical team members leaving and taking knowledge with them</td>
</tr>
<tr>
<td>8. By introducing simulation as a compliment to traditional training, you minimise risk of intimidating resistant trainers/trainee</td>
<td>8. Extra time and effort required during content creation stage to collaborate with all parties</td>
</tr>
</tbody>
</table>
Table 6: SWOT from Trainers Point of View

<table>
<thead>
<tr>
<th>SWOT from Trainers Point of View</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Weaknesses</strong></td>
</tr>
<tr>
<td>1. High level of Fidelity and Realism</td>
<td>1. Side Effects and Simulator Sickness</td>
</tr>
<tr>
<td>2. Safe and Control Training Environment</td>
<td>2. Not realistic enough to replace underground training</td>
</tr>
<tr>
<td>3. Create High level of Skill and Competency</td>
<td>3. Technology Compatibility</td>
</tr>
<tr>
<td>4. Overcoming Logistics constraints</td>
<td>4. Technology Constraints</td>
</tr>
<tr>
<td><strong>Opportunities</strong></td>
<td><strong>Threats</strong></td>
</tr>
<tr>
<td>1. Realistic enough to replace theory based classes</td>
<td>1. High Initial Investments</td>
</tr>
<tr>
<td>2. Training New comers</td>
<td>2. Side Effects</td>
</tr>
<tr>
<td>3. Opportunity of training all different scenario</td>
<td>3. Technology Constraints</td>
</tr>
<tr>
<td></td>
<td>4. Limited facilities equipped with this technology</td>
</tr>
</tbody>
</table>

CONCLUSIONS

As a conclusion VR is capable of overcoming real world training constraints and also fulfilling the gap between real life and traditional training approaches. It is necessary to realise that the VR training tool can complement traditional and practical training and will not replace them. However this research suggests that not all scenarios can be trained for using 360-degree VR, which has prompted Coal Services to develop Desktop VR.

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FIELD TESTING AND RELIABILITY ASSESSMENT OF VIDEO BASED FIRE DETECTION IN COAL MINING AND COAL HANDLING ENVIRONMENTS

Frank Mendham¹, David Cliff² and Tim Horberry³

Abstract: Previous laboratory test results and numerical modelling showed that Video Based Fire Detection (VBFD) offers a means of providing earlier fire detection compared with traditional carbon monoxide (CO) detectors in typical Australian underground mines. Additionally, from a reliability viewpoint compared with VBFD, CO detectors are subject to sensor drift and insensitivity as a result of contamination of the CO sensor device as reported by the National Institute of Occupational Safety and Health (NIOSH). The reliability of VBFD in an underground mining environment was unknown until field-testing as the final part of this research program was carried out.

An advantage of VBFD over traditional alternatives is that the camera lens does not have to be exposed directly to the products of combustion of a fire and can view the fire effects from a distance. Given the correct operational conditions to detect early smoke (or flame), including appropriate light and a suitable viewing arrangement, VBFD provides the required early detection needed to detect early smoke production from underground mine fixed plant fires. The question of whether the harsh environment that the VBFD cameras are exposed to, such as from coal dust and diesel fuel particulates has the potential to obscure the camera vision leading to a loss of VBFD detection capability and even unwanted alarm activation, was successfully answered. The results of the Arnot Power Station investigation indicated that if proper commissioning is carried out and effective maintenance is employed, a reliable means of early smoke (and flame) detection in underground mines is possible using VBFD.

This chapter concludes a three part research project in investigating and developing VBFD to improve fire life safety and asset loss control in underground mines. Overall, it found that VBFD is a more effective and more robust approach to providing timely fire detection and warning than traditional CO detectors in typical Australian underground mines.

INTRODUCTION

The effectiveness of Video Based Fire Detection (VBFD) to detect early smouldering fire has been previously experimentally compared with traditional Carbon Monoxide (CO) sensing under simulated mine conditions in a laboratory at the Safety in Mines Testing And Research Station (SIMTARS). It was found that VBFD reacted to very low level pyrolysis fires simulating overheated conveyor belt bearing housings, however CO sensing in most of the experiments recorded very minimal (or nil) levels of CO in sensor locations surrounding the fire source. Subsequent numerical modelling estimated similar results to the experiments, so the Computational Fluid Dynamics (CFD) was considered to validate the SIMTARS experiments. (Mendham et al., 2014a, 2014b).

The aim of the VBFD field study was to review and utilise the results of installation commissioning data and operational testing of a VBFD system in an operational underground mine environment. The purpose was to inform the underground mining industry of limitations that may exist in terms of the VBFD reliability in service, such as potential maintenance issues. It also sought to identify and address any possible inconsistent VBFD operation with respect to false positive alarm activation. Unfortunately,
one main factor influenced not being able to install a VBFD system in a working underground coalmine to facilitate this testing. During the period of the subject study into VBFD as a means of improving fire life safety and asset loss control in underground mining, the Australian resources industry moved from being in a position to readily facilitate and support minor external unfunded research projects within its underground mine assets to the current position where it is no longer willing to assist. As a result of the lack of a trial VBFD system in an underground mine, the subject field study opportunistically considered the reliability and performance of VBFD in a very similar and related environment, however this facility was not an underground mine.

The manufacturer of the VBFD equipment advised that a coal fired power station in South Africa had recently been fitted with an extensive VBFD system, including coverage in coal handling and storage areas (Mottle 2015). This location was considered an alternative to field-testing a VBFD system in an underground mine, as the coal handling systems are very similar and the level of airborne pollutants are also likely to be comparable.

Arnot Power Station is an asset of Eskom Holdings Limited located approximately 50 km east of Middelburg in Mpumalanga, in South Africa. Eskom advise that it was originally constructed in 1965 through 1966 and commenced operation in 1968. As a result of surplus power available at the time, Arnot was mothballed in 1992, but was recommissioned in 1998 due to growing power demands. Technical details (Eskom 2015) are as follows:

- Six (6) x 350MW generator units
- Installed capacity of 2,100 MW
- 2001 capacity of 1,980 MW
- Design efficiency at rated turbine Maximum Continuous Rating (MCR) (%): 35.60%
- Ramp rate of 34.48% per hour
- Average availability over last 3 years of 92.07%
- Average production over last 3 years of 9,675 GWh

Arnot mine is situated some 43 km from Middelburg in South Africa’s Mpumalanga province and feeds Arnot Power Station. This mine extracts coal using both underground and opencast operations, employs 1100 employees (with an additional 300 contract employees deployed in the opencast operation) with a run of mine of 1.44 Mt of thermal coal. Arnot uses mechanised mining methods and continuous mining processes. The mine is contracted to supply Eskom’s Arnot Power Station with coal on a “cost-plus” agreement in which there is a return on investment and a management fee. The mine has a coal reserve base of 54.2 Mt and a resource of 250.3 Mt. Coal is fed to Arnot Power Station by overland conveyor (Exxaro 2015).

The ongoing refurbishment works at Arnot incorporated a comprehensive fire detection system involving 360 commissioned VBFD cameras reporting to one control room using a Spyderguard™ system. C3SS Ltd. (C3SS) of Johannesburg, RSA, installed and commissioned the VBFD system between 2014 and 2015. It was noted during inspection that many areas in coal-fired power stations involving the use of fixed plant have similar functions and fire risks to underground coalmines. Reviewing the power station fire risks and its VBFD system performance in a location that contained many of the fire risks and types of fixed plant found in an underground mine was considered useful in generally assessing the potential reliability of VBFD in mining applications.

The relevance of using a power station as an alternative to an underground mine for the assessment of VBFD reliability and performance is that Arnot Power Station currently has one of the world’s largest and newest industrial VBFD installations throughout all areas of this large facility, including within its extensive coal handling and bunkering areas. Environmentally this scenario is very similar to a typical Australian underground coalmine and was considered an appropriate means of assessing VBFD reliability and performance at a test location. The assessment showed that ‘dirty’ camera housing
lenses, occurring as a result of coal dust deposits and similar pollutants potentially obscuring the fire detection scene, have little effect on detection capability up to a threshold alarm point that warns the operators of an impending reduction in detection performance unless the view is reinstated. Interestingly, the method of cleaning the lenses at Arnot was quite non-technical: a ‘feather duster’ is located at each VBFD detector and is applied on both infrequent occasions by operators in anticipation of an automatic threshold alert from the VBFD management system and on a scheduled basis. Whilst this may not be directly comparable for underground mines, it highlights the reliability and robustness of VBFD.

AIMS

The aim of the field-testing was to demonstrate whether VBFD can reliably and effectively be utilised in an underground coal-mining environment based on its operational performance in a similarly onerous environment, being a coal fired power station. The specific field-testing aim was to observe and report on two reliability related VBFD areas of interest also applicable to mining. These were:

1. VBFD Failure
2. VBFD False Response

Generally, VBFD failure in service could occur as a result of one of a number of possible failure modes. The following is a non-exhaustive list of such potential failure modes:

1. Failure to view potential fire risk due to contamination/obscuration of the VBFD lens or the external camera housing lens with particulate matter, such as coal or stone dust;
2. Failure of communications between the VBFD CCTV cameras and the control room;
3. Failure of the VBFD CCTV camera including power supply.

In the context of VBFD alarm inconsistency, which may be caused by false positive alarm stimuli, the following list is non-exhaustive and summarises inappropriate response modes:

1. Incorrect recognition of smoke-like phenomenon such as dust or steam as smoke;
2. Incorrect recognition of reflected light or direct light on the VBFD interpreted as smoke;
3. Discolouration of sections of conveyor belts in motion creating smoke-like appearance;
4. Inadequate or inappropriate illumination of the smoke plume;
5. Motor vehicle lights or cap lights shining on VBFD lenses or viewed sections of airborne particles (as per Item 2);
6. Motor vehicle and portable plant exhaust fumes;
7. Reflections from Personal Protective Equipment (PPE) such as reflective stripes and
8. Ingress of water into VBFD components.

METHOD

In order to focus the analysis of VBFD reliability and its performance at Arnot Power Station over a succinct investigation period, areas of the power station analogous to underground mines were identified for subsequent review. In the case of a typical coal-fired power station these areas were those used for the handling and storage of coal. The method utilised to identify these areas was to carry out a firsthand supervised inspection of the facility involving the recording of information about suitable locations where VBFD existed that were considered similar to underground mine scenarios. Additionally, photographs and diagrams were assessed on site to obtain a more thorough understanding of the power station layout and functioning and how VBFD is applied in each situation. After the identification of locations that were considered analogous to typical Australian underground coalmines, commissioning test results and progressive maintenance records for each applicable VBFD camera were reviewed. Alarm activity logs were then accessed to identify whether or not the alarm activations are likely to have occurred as a result of actual fires or as false positive activations.
In summary, the method utilised to collect VBFD reliability and performance data at Arnot Power Station included:

**Method 1:** Identification of locations analogous to underground coal mines and factors associated with the environment;

**Method 2:** Review of commissioning and maintenance data of VBFD in the identified locations to assess VBFD reliability;

**Method 3:** Reliability review of alarm activation logs to assess ‘Real versus False Positive’ activations, the identification of possible causes.

The methodology involved the review of actual VBFD images / recordings, which were analysed to assess the performance of commissioning tests and subsequent VBFD activations.

### RESULTS

VBFD locations analogous to underground mines included:

1. Coal Staithe Transfer Area; (Refer 1)
2. Coal Staithe conveyors; (Refer 2)
3. Inclined conveyors to coal bunkers; (Refer 3)
4. Coal staithe drive pulleys; (Refer 4)
5. Coal bunkers; (Refer 5)
6. Coal bunker walkways; (Refer 6)
7. Coal bunker conveyor offload areas; and
8. Coal bunker tail pulleys.

Clearly, numerous locations within Arnot power station closely resemble underground coalmines, so the relevance of VBFD assessment in these locations was found to be pertinent.

### Commissioning and maintenance

The commissioning of the VBFD system at Arnot Power Station was quite extensive, as it involved 360 cameras and was in accordance with the requirements of Fike Ltd, the equipment manufacturer (Privalov and Lynch 2012), the applicable standards of South Africa (SANS 10139, 2012) and international standards (NFPA 72, 2015). Except for the South African national standard, these prescriptive requirements also apply to Australian underground mines from both a regulatory and insurer perspective. One significant point of interest is that the commissioning tests listed in these standards are less onerous than the laboratory tests previously carried out at SIMTARS by the first author. The VBFD tests at SIMTARS required much smaller smoke plumes to be accurately detected than those stipulated in the standards. The VBFD installation and commissioning standards require commercially available ‘90 second Smoke Emitter Candles’ to be used as the source of smoke such that the VBFD must be able to detect the simulated smoke generated by the candle. The VBFD does not necessarily have to operate within the 90-second smoke emitter discharge period, as the residual smoke subsequent to the extinguishment of the candle may be required to spread from its emitter source to within the programmed field of view of the VBFD camera.

The installers and commissioners of the VBFD system (Grange 2015) emphasised the requirement to commission it in accordance with the manufacturers’ specifications and with the appropriate standard to ensure that the installation will operate as intended without unexpected outages.

The required commissioning tests were:

1. VBFD field of view is clear of all obstructions not usually in view;
2. The camera image quality is adjusted to optimum level;
3. Confirm no physical damage to the VBFD camera or wiring;
4. Field Of View (FOV) is set up top include roof (ceiling), target image and floor;
5. Content Fault (for smoke detection only);
6. Network Fault (if Applicable);
7. Power Loss; and
8. Analytics Functioning.
VBFD images may be divided into various zones for a number of purposes, such as facilitating identification of smoke spread in a space, or activation of suppression systems at specific fire zones. Zones of a VBFD screen view can be programmed to ‘do nothing’, that is, be excluded from the VBFD surveillance. VBFD exclusion zones are sections of the image that can be isolated from the smoke (or flame) detection process so that if smoke like phenomenon is expected to occur in these excluded areas, it does not initiate an alarm.

Advice was received in relation to the field of view of the VBFD cameras. The installers (Grange, 2015) experience was that the VBFD cameras more effectively detect smoke when the field of view includes the ceiling and floor as well as the potential target fire source. Grange further explained that it is imperative that an appropriate amount of time be taken to fine-tune the VBFD ‘exclusion zones’.

### VBFD Reliability

Periodically, where intense floor and equipment cleaning is carried out involving high pressure water cleaners producing considerable dust plumes that resemble smoke plumes and water vapour movement involving unstable changes to ambient light levels through the introduction of specialised relocatable work lighting – VBFD alarm activations had occurred as shown in Table 1.

<table>
<thead>
<tr>
<th>Category of VBFD Issue</th>
<th>Frequency Per Year</th>
<th>Typical Cause</th>
<th>Typical Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>VBFD False Response</td>
<td>Five (5) per 360 VBFD units</td>
<td>Plant cleaning activities. (Typically: high-pressure water cleaning).</td>
<td>Isolate VBFD until maintenance activities completed. Operators instructed to advise prior to carrying out next high-pressure water cleaning activity.</td>
</tr>
<tr>
<td>VBFD Failure</td>
<td>Two (2) per 360 VBFD units</td>
<td>Obstacle placed in FOV. (Typically: portable crane or gantry in FOV).</td>
<td>Isolate VBFD until obstacle removed.</td>
</tr>
</tbody>
</table>

The contractor (Grange, 2015) explained that VBFD has limitations in terms of discriminating the ‘signature’ of smoke from phenomenon that appears smoke-like. These limitations need to be taken into account by operators and those carrying out activities that are similar in nature to creating smoke. The advantage of VBFD over other forms of smoke (or flame) detection is that VBFD provides the opportunity for manual intervention by the control room through the ability to actually view the potential fire on-screen. Of course, in the absence of manual intervention (e.g. unstaffed control room) the VBFD system may be configured to continue the fire recognition process automatically to generate a fire evacuation cue or initiate automatic fire suppression.

The developing capability of VBFD to accurately detect and discriminate fire phenomenon from other smoke-like phenomenon is managed under the VBFD developer’s continuous improvement processes. Additionally, maintenance activities need to be carried out to ensure the VBFD system can operate effectively without being impacted by reduced field of view through dust build-up on the lens and reduced illumination.

If not managed, dust contamination of the VBFD camera lenses will cause them to fail to detect a source of smoke, however this is countered by the VBFD analytics generating user programmable warning alarms to the controllers advising that lens obscuration is nearing a threshold and maintenance intervention (cleaning) of the VBFD camera lens or the camera-housing lens is required. All VBFD
camera lenses are ‘dry’ cleaned at Arnot Power Station using a domestic feather duster applied every two (2) weeks. An image showing the level of surface dust accumulation on VBFD cameras in typically ‘dirty’ locations is provided in Figure 7 whilst the typical view from a camera with an obscured lens achieving 55.79% visibility, is shown in Figure 8.

Figure 7: Typical Dust Accumulation on VBFD CCTV Camera

Figure 8: Obscured view from VBFD camera with 55.79% (Dirty Lens) visibility

The VBFD installation firm (C3SS) advised (Grange, 2015) during the site interview that there had been no VBFD failures specific to the 360 installed CCTV cameras, however some minor outages had been encountered due to work by others causing loss of communications through cabling disturbance, such as disconnections of fibre optical cables without proper notice. These instances were system failures rather than failures specifically associated with the VBFD installation.

DISCUSSION AND CONCLUSIONS

The performance and reliability of VBFD at a coal-fired power station in South Africa was reviewed, because this installation has many similarities in its operations to typical underground coalmines, so parallels could be drawn from this in the absence of an available underground mine VBFD system. These similarities are clearly evident in coal conveyor belt systems, transfer points and coal bunkering facilities.

At Arnot Power Station these areas have VBFD implemented throughout, as might similarly be the case in an underground coalmine. Arnot is the largest VBFD installation worldwide with approximately 360 VBFD cameras installed, as reported by Fike, the VBFD manufacturer (Mottley, 2015). Arnot power station was a source of considerable VBFD reliability data representing knowledge that could be transferred to VBFD installations used in underground mining in Australia.

This investigation has shown that VBFD systems can effectively be incorporated in locations that are affected by airborne pollutants including coal dust, water mist and mist hydrocarbons. It was identified that it is essential that the VBFD systems be properly installed and commissioned in accordance with
regulatory requirements and in particular, manufacturers specifications. Commissioning adjustment will involve the careful observation and subsequent blanking out of potential sources of false activation of the VBFD.

VBFD systems require a level of maintenance specific to their environment. In the power station they required dry cleaning of the lenses on a regular basis and in underground mines it is likely they will require a similar level of attention. Future research in relation to VBFD reliability might specifically identify ways of ensuring camera and housing lenses require very little cleaning and maintenance and this could be as a result of self cleaning lenses that incorporate external air supplies that prevent dust deposits on the lens surface.

The results of the Arnot Power Station investigation indicated that if proper commissioning is carried out and effective maintenance is employed, a reliable means of early smoke (and flame) detection in underground mines is possible using VBFD.

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SELF-HEATING BEHAVIOUR OF HEAT-AFFECTED COAL

Basil Beamish¹, Mark Cosgrove² and Jan Theiler¹

ABSTRACT: Adiabatic oven testing of heat-affected and normal coal from the same location at Mandalong Mine shows substantial differences in intrinsic self-heating rates and self-heating behaviour. While initial self-heating rates of the heat-affected coal are greater than normal coal under site conditions, there is no sustained self-heating to thermal runaway due to a decrease in the number of reactive sites remaining in the heat-affected coal and greater inaccessibility to those sites that do exist. This behaviour is attributable to the vesicular nature of the heat-affected coal contributing to easy access to open pores resulting in the initial burst of self-heating. However, the thermal alteration of the coal also contributes to destruction of reactive sites and makes access to the remaining reactive sites in the micropore system of the coal more difficult.

INTRODUCTION

Adiabatic oven testing has been used routinely by Australian and New Zealand coal mine operations since the early 1980’s to rate the propensity of coal to spontaneously combust (Humphreys et al., 1981). Many different features of coal have been studied using this technique to establish their affect on self-heating rates. These include: changes in coal rank (Beamish and Arisoy 2008 and Beamish and Beamish 2012); coal type (Beamish and Clarkson 2006); and mineral matter composition (Beamish and Blazak 2005 and Beamish and Sainsbury 2008).

As coal rank increases due to the natural coalification process the coal changes both chemically and physically and this results in a decrease in the intrinsic self-heating rate of the coal. Some seams can come into contact with or are adjacent to igneous intrusions, and under these circumstances the coal is artificially rank advanced through the devolatilisation that takes place in response to the exceedingly high temperatures experienced by the coal (Ward et al., 1989; Kwiecinska and Petersen 2004 and Singh et al., 2007). This results in what is commonly referred to as heat-affected or coked coal. The self-heating behaviour of such coal is not reported in the literature and this paper presents the results of recent self-heating tests of both heat-affected and normal coal from the same location of the Mandalong Mine in New South Wales.

COAL SAMPLES AND ADIABATIC TESTING

Within the general vicinity of Main gate19 at Mandalong Mine a localised igneous sill has penetrated the upper part of the seam and reaches a thickness of roughly 80 cm. Approximately 1.5 m of heat-affected coal overlies the sill. This coal will be caving into the goaf as the longwall panel extracts the coal from below the sill. It is therefore important to establish the spontaneous combustion propensity of the heat-affected coal in comparison with the normal coal from the same area to ensure appropriate management practices are in place. In addition, due to the expected rank increase of the heat-affected coal due to thermal alteration, a high rank bituminous coal from the Goonyella Lower Seam is used in this paper to compare the self-heating behaviour.

The coal quality details of the samples are contained in Table 1. The normal Mandalong coal is high volatile A bituminous in rank and the heat-affected coal has decreased in volatile matter from 35.9% to 5.1%, which creates an artificial rank elevation equivalent to anthracite. The heat-affected coal also shows an increase in ash content due partially to concentration of the existing mineral matter as the coal

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devolatilised and injection of extraneous mineral matter from the intrusive fluids accompanying the sill emplacement.

Samples of both normal and heat-affected coal from MG19 have been tested using an adiabatic oven to establish their R70 values and to benchmark the time taken to reach thermal runaway (incubation period) under conditions more closely resembling those of the mine site. The R70 testing procedure is described by Beamish (2005) and essentially involves testing a dried, crushed coal sample under adiabatic conditions from a fixed starting temperature of 40°C. The incubation test procedure, known as SponComSIM™ testing, uses the coal in its as-mined moisture state from a starting temperature that reflects the site-specific conditions. The results obtained provide both an indication of the time taken to reach thermal runaway and the characteristic behaviour of the coal as self-heating progresses. This can be compared against case history coals of known self-heating behaviour as well as mine site experience.

Table 1: Coal quality data for coal samples from MG19, Mandalong Mine and high rank Goonyella Lower Seam

<table>
<thead>
<tr>
<th></th>
<th>Mandalong Normal Coal</th>
<th>Heat-affected Coal</th>
<th>Goonyella Lower Seam</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PROXIMATE ANALYSIS</strong> (air-dried basis)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture (%)</td>
<td>2.2</td>
<td>2.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Ash (%)</td>
<td>18.5</td>
<td>28.2</td>
<td>21.9</td>
</tr>
<tr>
<td>Volatile Matter (%)</td>
<td>29.5</td>
<td>5.7</td>
<td>17.2</td>
</tr>
<tr>
<td>Fixed Carbon (%)</td>
<td>49.8</td>
<td>63.6</td>
<td>59.7</td>
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<tr>
<td><strong>ULTIMATE ANALYSIS</strong> (dry ash-free basis)</td>
<td></td>
<td></td>
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<tr>
<td>Carbon (%)</td>
<td>82.3</td>
<td>91.2</td>
<td>86.9</td>
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<tr>
<td>Hydrogen (%)</td>
<td>4.98</td>
<td>2.17</td>
<td>5.00</td>
</tr>
<tr>
<td>Nitrogen (%)</td>
<td>1.68</td>
<td>1.75</td>
<td>1.78</td>
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<tr>
<td>Sulphur (%)</td>
<td>0.38</td>
<td>0.19</td>
<td>0.56</td>
</tr>
<tr>
<td>Oxygen (%)</td>
<td>10.7</td>
<td>4.7</td>
<td>5.8</td>
</tr>
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</table>

**COAL RANK PARAMETERS**

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
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<td>Volatile Matter (%)</td>
<td>35.9</td>
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<td>20.4</td>
</tr>
<tr>
<td>Calorific Value</td>
<td>14061</td>
<td>na</td>
<td>na</td>
</tr>
<tr>
<td>ASTM rank</td>
<td>hvAb</td>
<td>an</td>
<td>lvb</td>
</tr>
</tbody>
</table>

na – not applicable; hvAb – high volatile A bituminous; lvb – low volatile bituminous; an – anthracite

Samples of both normal and heat-affected coal from MG19 have been tested using an adiabatic oven to establish their R70 values and to benchmark the time taken to reach thermal runaway (incubation period) under conditions more closely resembling those of the mine site. The R70 testing procedure is described by Beamish (2005) and essentially involves testing a dried, crushed coal sample under adiabatic conditions from a fixed starting temperature of 40°C. The incubation test procedure, known as SponComSIM™ testing, uses the coal in its as-mined moisture state from a starting temperature that reflects the site-specific conditions. The results obtained provide both an indication of the time taken to
reach thermal runaway and the characteristic behaviour of the coal as self-heating progresses. This can be compared against case history coals of known self-heating behaviour as well as mine site experience.

**ADIABATIC TESTING RESULTS AND DISCUSSION**

**R70 values and intrinsic self-heating rate behaviour**

The R70 self-heating rate curves for the normal and heat-affected coal samples from MG19 are shown in Figure 1. The normal coal has an R70 value of 2.12 °C/h, which rates the coal as having a medium intrinsic spontaneous combustion propensity (Beamish and Beamish 2012). However, the heat-affected coal initially self-heated before the temperature levelled out at 62 °C and did not reach the standard 70 °C that is used to define the R70 value of the coal. Therefore, the R70 value of the heat-affected coal is recorded as 0.

Figure 1 also contains the self-heating curve for a high rank bituminous coal sample from the Goonyella Lower Seam. It can clearly be seen that the heat-affected coal has an initial self-heating rate that is faster than the Goonyella Lower Seam sample, but the two self-heating curves cross over after about 40 hours on test. Subsequently, the Goonyella Lower Seam sample proceeds to thermal runaway, whereas the heat-affected coal sample stalls. This is presumably related to a combination of both physical and chemical difference between the two coal samples. Heat-affected coals contain vesicles in the form of pits and cavities of variable sizes due to the escape of volatiles (Singh et al., 2007). This enables easy access of oxygen to available reactive sites initially. However, a majority of the original reactive sites are destroyed due to the thermal alteration process from the igneous intrusion and the remaining reactive sites are contained in the much finer micropore system of the heat-affected coal, which has undergone significant annealing thus reducing the accessibility to these remaining reactive sites.

The heat-affected sample was step-heated to just over 100 °C to compare against the exponential self-heating rate of the Goonyella Lower Seam sample from this same temperature. The results are shown in Figure 2. Even at elevated temperatures the self-heating rate of the heat-affected coal is quite slow and there is no major rise in the self-heating rate until the coal temperature exceeds approximately 150 °C.

![Figure 1: Adiabatic R70 self-heating curves for normal and heat-affected coal from MG19, Mandalong Mine compared against high rank bituminous coal from the Goonyella Lower Seam](image-url)
Figure 2: Comparison between self-heating rates of heat-affected coal from MG19 and Goonyella Lower Seam coal at elevated temperatures

Incubation testing and self-heating behaviour

The SponComSIM™ test results for the heat-affected and normal coal from MG19 are shown in Figure 3. The initial self-heating rate of the heat-affected coal is greater than the normal coal, which again can be attributed to the open pore structure (vesicular nature) of the heat-affected coal. However, after reaching a maximum temperature of approximately 38 °C, the temperature of the heat-affected coal gradually decreases due to the overriding influence of moisture evaporation combined with hindered access and a decrease in the number of reactive sites. It is clear from these results and the results of the R70 self-heating rate testing that the heat-affected coal is not capable of reaching thermal runaway, unless it is exposed to a significant external heat source. Even then, it would take a considerably long time for the coal to reach thermal runaway. If the normal coal was present in a loose pile of critical thickness with sufficient continuous air supply and minimal heat dissipation, then spontaneous combustion incubation would be possible to elevated temperatures. However, this is strongly mitigated by the goaf inertisation practice in use at Mandalong, which utilises methane from the in-seam gas drainage system to inert the goaf atmosphere (Claassen 2011).

CONCLUSIONS

The results of adiabatic self-heating tests from normal and heat-affected coal, once again show that coal self-heating performance is not a simple predictable behaviour. Under ideal site conditions, the heat-affected coal is capable of initially self-heating above ambient temperatures, but due to the combined effects of moisture evaporation as well as limited access and the decrease in the number of reactive sites, sustained self-heating is not possible without the presence of a significant external heat source. The physical and chemical changes to the coal created by thermal alteration from an igneous sill have contributed to the self-heating behaviour observed. From a risk management perspective the caving of this coal into the goaf does not therefore create any additional risk of creating a spontaneous combustion event.
Figure 3: Adiabatic SponComSIM™ self-heating curves for normal and heat-affected coal from MG19, Mandalong Mine

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REFERENCES


INTERPRETING THE STATUS OF AN UNDERGROUND COAL MINE HEATING FOR VALID RISK MANAGEMENT AND CONTROL

Brendan Newham¹, Basil Beamish², Michael Brady³ and Matthew Fellowes⁴

ABSTRACT: The status of an underground coal mine heating event is predominantly assessed based on gas sampling results from the affected area. The gas samples can be obtained in a number of different ways, but for accurate determination of the gases present analysis is normally performed using gas chromatography. Several gas indicators have been identified from past experiences that are used in combination to determine a Trigger Action Response Plan for individual mines. The upper levels of the Trigger Action Response Plan contain more urgent actions in response to the assessed level of advanced heating, with the ultimate level being withdrawal from the mine. Once this point is reached the only action that can be taken to control the event has to be done remotely and also, once activated it is difficult to re-enter the mine. Recent experience of a heating event provides new data that demonstrates the importance of understanding and interpreting the status of hot spot development to risk management and control applied by an Incident Management Team. This paper discusses the use of gas trending, including the development of: a new gas indicator ratio that is very sensitive to the heating status and control measures applied; and a hot spot tracking diagram that provides a good visual representation of the heating status and the response to actions taken.

INTRODUCTION

The identification of a spontaneous combustion event in an underground coal mine relies on vigilant use of a valid Trigger Action Response Plan (TARP) that has been designed to match the likely spontaneous combustion behaviour of the coal being mined. The TARP contains trigger levels based on a combination of physical (visual) signs and gas or gas ratio indicators that are chosen to enable appropriate early responses to be implemented to manage and control the event. At a more advanced stage of an event it may be necessary to prepare for sealing of the area to exclude air ingress, or in a more extreme case to completely inert the sealed area by injecting for example nitrogen. Once an event has been identified there is also the need for appropriate decision making to be implemented by an Incident Management Team (IMT) based on sound risk management supported with valid data interpretation of the status of the “heating event”. This decision making process is very important as the uppermost level of the TARP is withdrawal from the mine, which has consequences for both safety of mine personnel and the management options available once this point is reached.

This paper presents a case study of a spontaneous combustion event that occurred in 2015, which was successfully managed and controlled by applying sound engineering principles. Key stages of the event are discussed in the paper rather than a blow by blow description of all the actions taken. As a result of the event some new lessons were learned that may assist in improving the management of future events.

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BACKGROUND TO THE SPONTANEOUS COMBUSTION EVENT

On Thursday 25th June 2015 a “heating event” was confirmed in 18 CT of the West Headings in the Great Northern Seam (GNS) at Mannering Colliery (Figure 1). This discovery resulted after a lengthy and difficult search initially triggered by a low level carbon monoxide alarm spike from the tube bundle monitor point at the GNS entry to the upcast shaft of the mine. The indicators found at the site included heat haze, sweating and a “benzene” smell, with elevated levels of carbon monoxide recorded by hand held gas detectors. Prior to this event, there were no known recordings of spontaneous combustion occurring at Mannering Colliery throughout its more than 50-year mine life. In addition, there were very few known recorded occurrences of spontaneous combustion in the Great Northern Seam workings throughout its extensive mining history, although spontaneous combustion events have been known to occur in stockpiled coal on the surface.

![Figure 1: Location plan of heating site](image)

The depth of cover to the Great Northern Seam is generally in the range of 150-210 m and there is typically around 30 m of interburden between the Great Northern Seam and the underlying Fassifern Seam. In-situ gas contents range from 1.6-2.75 m3/t for the Great Northern Seam and 2.4-3.5 m3/t for the Fassifern Seam. The seam gas composition is predominantly methane (>95%). R70 self-heating rate test results indicate an intrinsic spontaneous combustion propensity rating of medium to high for the Great Northern Seam, dependent on the ash content. However, the incubation period for this coal is strongly dependent on a number of site specific parameters and other coal properties.

The mine ventilation arrangements in place for more than the past two years, since placing the mine on care and maintenance, consists of a reduced flow of approximately 130 m3/s at a collar pressure of around 600 Pa. Typically, much of the underlying Fassifern Seam workings ventilation air returns up through inter-seam staple shafts and flood ventilates the old Great Northern Seam workings prior to returning to the upcast shaft. The mine has five inter-seam staple shafts that are still in use to some extent. Very few areas of the mine, either in the Great Northern Seam or the Fassifern Seam are effectively sealed. This is typical of the district, which has been mined for in excess of 100 years.
The mine uses a Maihak 10 point tube bundle monitoring system. The bottom of each of the staple shafts is monitored to capture all Fassifern Seam workings return air just prior to entering the Great Northern Seam workings. In addition, five other fixed locations are monitored within the Great Northern Seam workings, including the top of the upcast shaft, which captures the total mine return air. Mine atmosphere monitoring also includes the Mining Supervisor underground inspections and hand held monitoring. Bag samples are typically taken monthly from the top of the upcast shaft.

**GAS MONITORING RESPONSE TO THE “HEATING EVENT”**

Initial data and interpretation of the “heating event” status

On Friday 26th June 2015 an IMT was formed consisting of the Mine Manager, the Ventilation Officer and the General Manager of Mines Rescue and Regulation and Compliance. A number of actions were implemented including the mobilisation of the Coal Mines Technical Services (CMTS) gas chromatograph to be onsite as soon as possible as well as planned underground activities to manage and control the incident. All formal notifications had been issued the day before as required by the WH and S (Mines) Act 2013 Part 3, WH&S (Mines) Regulation 2014 Cl 179(d): “any initial indication that any underground part of a coal mine is subject to windblast, outbursts or spontaneous combustion”.

The IMT were keen to establish the status of the “heating event” as this would have implications for the management actions required to control the event. The IMT contacted an external consultant who had previous experience and knowledge of the self-heating behaviour of the Great Northern Seam coal to place the initial gas readings taken the day before and the subsequent gas monitoring results into perspective. Subsequently, an additional consultant with gas monitoring experience was engaged so that the two consultants could work as a team and report back to the IMT on interpretation of the “heating event” gas monitoring results in response to control actions as they were implemented. At 6:30pm tube bundle sample point #10 was set up on its own purge pump at location 4 shown in Figure 1 and bag sampling commenced every hour, processed via the newly setup gas chromatograph and reported using SmartMate software.

The initial downstream gas readings of the “heating event” at location 4 (Figure 1) indicated a Graham’s ratio of 2.54, with ethylene present. The general information in the SmartMate software indicated that for European coals this would correspond to a serious heating. It is also noted that all mines should establish their own levels and that older coals can produce a higher Graham’s ratio. None of the literature values take into consideration the status of hot spot development.

In 2004, bulk self-heating tests were conducted on coal from Mandalong Colliery in both a gassy as-mined state and non-gassy, dried coal state (Jabouri 2004 and Beamish and Jabouri 2005). The coal tested is analogous to that present at Mannering Colliery (same seam and only 8 km away). The relationship between hot spot coal temperature and Graham’s ratio from these tests (Figure 2) indicates that for the Graham’s ratio of 2.54 the hot spot temperature is in the range of 110-130 °C, depending on how dry the coal is. In the area of the “heating event” site the coal had been standing for a considerable number of years. Therefore the hot spot temperature was more likely to be closer to the lower end of the temperature range indicated by the Graham’s ratio. In addition, this was only a spot reading and subsequently 53 regular readings from 6:30pm 26/6/2015 to 5:56pm 28/6/2015 yielded a Graham’s ratio of 2.21±0.10.

A review of the data obtained by Jabouri (2004) from bulk self-heating tests of Mandalong coal showed that at the hot spot temperature indicated by the Graham’s ratio, the ethylene/ethane ratio becomes a good confirmation indicator of the hot spot development status (Figure 3). When this ratio is 0 it is an indication that the hot spot has not advanced to a significant stewing stage. Once the value increases above 0.3, a well-defined hot spot has formed and the hot spot begins to migrate towards the free surface. The faster this ratio approaches 1 the more rapid the temperature increase becomes with
ignition being imminent. This trend in the ethylene/ethane ratio was also confirmed by additional bulk self-heating test results for Mandalong coal obtained from testing conducted in 2010 (Pantano, 2010). The initial bladder sample produced an ethylene/ethane ratio of 0.12. When compared to the results of the bulk self-heating tests of Mandalong coal this indicated a hot spot temperature range of approximately 100-120 °C (Figure 2). The slightly lower temperature range indicated by the ethylene/ethane ratio compared to the Graham’s ratio may be an artefact of minor ethane seam gas being added to the airstream and thus lowering the ethylene/ethane ratio. Fifty three subsequent readings yielded an ethylene/ethane ratio of 0.12±0.01.

Using the ethylene/ethane ratio removes the risk of missing hot spot temperature acceleration due to the dependence on gas indicators obtained from a single gas such as carbon monoxide make for example. An error in the measurement of carbon monoxide due to drift in calibration could also result in a false higher or lower Graham’s ratio.

Figure 2: Relationship between hot spot temperature and Graham’s ratio for Mandalong coal (original data from Beamish and Jabouri 2005)

Figure 3: Relationship between hot spot temperature and ethylene/ethane ratio for Mandalong coal (original data from Beamish and Jabouri, 2005)
Once the hot spot status had been identified it was possible to provide a more realistic risk assessment to amend the TARP to make use of logical trigger levels. These were assessed by considering the rate of increase in both the Graham’s ratio and ethylene/ethane ratio with temperature. Looking at the sensitivities of these two indicators, it was decided to implement a Graham’s ratio of 3 and an ethylene/ethane ratio of 0.24 for the Level 3 TARP, as these would indicate a substantial elevation in the hot spot temperature and coincide with the hot spot advancing towards a free surface.

**Hot Spot Tracking diagram and application to the “heating event” management and control**

Having decided on the best gas indicators to monitor the progress of the heating taking place there was a need to present this information in such a manner so that any changes taking place could be readily identified and matched against actions taken to manage and control the “heating event”. An additional factor was added to this tracking sequence, namely the barometric pressure in case changing natural conditions affected the ventilation and air supply to the heating.

Initially separate plots were used for each indicator with a timeline on the X-axis. However, as the event progressed it became apparent that a composite plot was needed for tracking the hot spot development and the response to ventilation changes. Initially, the Graham’s ratio was plotted on the left Y-Axis (with a range of 0 to 3, matching the Level 3 TARP trigger) and the barometer was plotted on the right Y-Axis. The values of the ethylene/ethane ratio were approximately 0.12 and the Level 3 TARP trigger had been set at 0.24. It was decided that to enable the Graham’s ratio and the ethylene/ethane ratio to be on the same plot a multiplier of 10 should be applied. This composite plot can be termed a Hot Spot Tracking (HST) diagram (Figure 4).

![Figure 4: Hot spot tracking response from 26/6/2015 to 1/7/2015](image)

Between the commencement of monitoring on the 26/6/2015 and early morning of 30/6/2015 both the Graham’s ratio and the ethylene/ethane ratio remained relatively stable even with a falling barometer (Figure 4). This enabled preparatory work to continue unhindered with the eventual aim of sealing off the area. However, by Tuesday afternoon (30/6/2015) after a recalibration of the Gas Chromatograph CO detector it became apparent that the baseline level of the Graham’s ratio had increased significantly and the ethylene/ethane ratio was increasing substantially (Figure 4). By late Tuesday evening these values were rapidly approaching the Level 3 TARP triggers.
An assessment of possible explosive mix scenarios had been performed throughout the day based on the makes of the various gases and oxygen consumption rate. Given the status of seal-up preparations at that time the only option available to control the air reaching the hot spot was to lower the previously installed rolled up flexible stopping at location 3 (Figure 1) and restrict the ventilation entering the heating site in preparation for a final sealing at location 4 (Figure 1) on the downstream side of the heating. The response to this ventilation control was expected to be a decrease in the gas indicator ratios and a decrease in the oxygen concentration.

The response from the implementation of the ventilation intervention of completing stopping #3 was almost immediate with a rapid drop in the ethylene/ethane ratio recorded within approximately three hours. A drop in the Graham’s ratio was delayed by almost 12 hours. The on-going drop in both the Graham’s ratio and the ethylene/ethane ratio provided confidence in being able to take additional time to organise the final seal-up of the area, particularly as no explosive mix was developing and the oxygen content began to drop noticeably. This was also supplemented with nitrogen injection into the area. Note the CO/CO2 ratio was also added to the HST diagram (Figure 5) at this stage to provide further confirmation of the success of the ventilation intervention. A scaling factor of 60 was applied to bring the value into the same Y-axis range.

A similar response in the gas indicators from restricting the ventilation to the incident area was also recorded for a spontaneous combustion event at Spring Creek Mine (New Zealand) in 2008 (Beamish and Hughes, 2009). The assessed hot spot temperature of that event was lower than this one, but the immediate response of the ethylene/ethane decrease was identical. The Graham’s ratio response was also delayed.

All gas indicator readings continued to decrease after Stopping #3 was completed and approximately two and a half days later the final seal at Stopping #4 was completed with additional nitrogen injection. The Graham’s ratio initially increased, but then gradually decreased over time following the seal-up (Figure 5). However, the ethylene/ethane ratio continued to progressively decline indicating that the hot spot was diminishing. Continued monitoring of the sealed area atmosphere recorded the oxygen content decreasing to below 3% and the gas indicator ratios continued to decline, indicating the event was successfully controlled.

Figure 5: Hot spot tracking response from commencement of event to seal-up
CONCLUSIONS

When managing an underground coal spontaneous combustion event it is imperative that the self-heating characteristics of the coal are known so that informed decisions can be made by the IMT based on the understanding of the status of the heating. A recent “heating event” at Mannering Colliery illustrates the practical advantages of implementing this sound engineering practice. Initial detection of the “heating” indicated the presence of a hot spot that was able to be interpreted from valid laboratory data as being in a semi-stable stewing stage between 110-130 °C. This enabled preparatory work to continue towards sealing the heating in a controlled manner. A new gas indicator ratio, ethylene/ethane was also able to be introduced as confirmation of the hot spot status, which proved to be more sensitive to control measures than the normally used Graham’s ratio.

Presentation of gas monitoring data to assess cause and effect was identified as crucial to interpretation of the “heating event” progress to assist with valid risk assessment for management and control. This was achieved by developing a composite plot that contained data for barometric pressure, Graham’s ratio and ethylene/ethane ratio. A scaling factor was applied to the ethylene/ethane ratio to enable it to be plotted on the same Y-axis as the Graham’s ratio. This new plot was termed a HST diagram as it provided a clear indication of hot spot status and progress as well as the response to control measures adopted. It was also shown that other useful gas indicators such as CO/CO₂ can be added to this diagram with appropriate scaling factors.

Staged control of the “heating event” was achieved by sequential ventilation restriction and sealing of the area. The rapid drop in gas indicators such as ethylene/ethane ratio shows how sensitive hot spot development is to ventilation control.

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The authors would like to thank Lake Coal for granting permission to publish these results.

REFERENCES


TRUCK AND SHOVEL VERSUS IN-PIT CONVEYOR SYSTEMS: A COMPARISON OF THE VALUABLE OPERATING TIME

Isaac Dzakpata, Peter Knights, Mehmet Siddik Kizil, Micah Nehring and Saiied Mostafa Aminossadati

ABSTRACT: Shovels, haul trucks and conveyors are used in surface mines for material haulage of which trucks are have been most widely used. This paper presents a comprehensive comparison study in terms of the operating efficiency of trucks and conveyors as applied in surface mining operations. Three key time usage metrics are used to assess the efficiency of both haulage systems namely: Utilised Time, Operating Time and Valuable Operating Time. The notion that measurement of equipment performance should focus on a multi instead of a single-factor approach is proposed. Comparison of the two systems based on these measures indicates that although trucks lend themselves to high flexibility and lower upfront capital outlay, conveyor haulage offers a better measure of performance on all the three metrics of measuring equipment performance. This opportunity lies in the high Valuable Operating Time achievable with the conveyors compared to the trucks. The results of analysis on 308,912 load records and 12 months equipment usage indicates that while the truck fleet achieve much higher Available Time and Utilisation Time, the conveyors achieve a higher Valuable Operating Time (25% higher) compared to the truck fleet. The conveyors achieved an average of 3,509 hours in Valuable Operating Time compared to the average Valuable Operating Time of 2,638 hours for the truck fleet. The inference made from this observation is that, the higher effective utilisation of a continuous haulage system for example In-Pit Crusher Conveying (IPCC) could significantly improve the operating efficiency of the loading equipment.

INTRODUCTION

Surface mine material haulage costs constitutes nearly 50% of the total mining cost (Nel et al., 2011; Thompson 2005; Kennedy 1999 and Bozorgebrahimi 2004). Haul truck and conveyor haulage are a major part of surface mine material haulage over the years, of which trucks have been most widely used (Burt and Caccetta 2014 and Radlowski 1988). It has been argued that trucks and shovels are common to 95% of the global surface mining fleet due to their flexibility and large economy of scale (De Lemos Pires 2013). Over the last three decades, conveyor haulage has gathered a great deal of momentum in association with In-pit Crusher Conveying (IPCC), which is argued to be a cost-effective alternative to the truck and shovel material handling system. With current trends in global mining industry (e.g. fluctuating fuel prices, lower commodity prices, high operating cost) coupled with the maturing of many surface operations, the efficiency of mining equipment has come to the forefront of discussion regarding the sustainability of surface mining. In addition, many mine operators are demanding more from existing assets as capital expenditure stalls. This calls for the ability to measure the overall efficiency of mining equipment for the purposes of improving the operating efficiency of mining operations, effective asset utilisation, resource scheduling, operational planning and resource optimisation. Figure 1 shows a schematic of a typical truck-shovel operation from loading area to dump location (ore or waste).

This paper compares the performance of truck-shovel and conveyors as applied in surface mining operations. The science of measuring the performance (productivity and effectiveness) of mining equipment has evolved and matured particularly for trucks and shovels. Useful metrics have been developed by Original Equipment Manufacturers (OEMs), mine operators and academics. Consistent with the recent trends toward increased asset optimisation and productivity across the industry (Ernst et al., 2014 and 2015), this aims to draw on the experiences gained from truck haulage in comparison with
the performance of in-pit conveying haulage. This study examines the performance of truck-shovel and conveyor operations based on a combination of the following three key time usage metrics (indicators):

Utilised Time (UT), Operating Time (OT), Valuable Operating Time (VOT)
In this study, the overall emphasis is placed on the VOT for each operation.

Figure 1: Schematics of (a) Truck – Shovel Operation (adapted: www.oilsandstoday.ca) and (b) In-pit conveyor in used a Fully Mobile Crusher and Shovel (Humphrey et al., 2011)

CURRENT APPROACHES TO MEASURING MINING EQUIPMENT PERFORMANCE

Lord Kelvin was the first in 1883 to recognize that one cannot improve something which cannot be measured. A review of literature supports the view that measuring the performance of an asset must not only capture the physical productivity but also the financial productivity (United States Bureau of Labor Statistics, 1988). Productivity relates outputs to inputs while effectiveness is the ratio of actual output to rated (or best). Business improvement practitioners are often more focused on productivity. Efficiency is widely expressed as a percentage of actual output to the maximum achievable output which is therefore a strong measure of how well an asset (in this case equipment) is performing with respect to best practice. Figure 2 shows a Time Usage Model (TUM) commonly used across the mining industry.

Figure 2: Example of Time Usage Model common to mining industry (Pintelon and Muchiri 2006 and Jeong and Philips 2001)

A critical review of literature indicates that while there is a measure of VOT, it is mostly applied to fixed plant and not mobile equipment. In mining operations, trucks are classified as mobile equipment whereas conveyors are classified as fixed plant as they are comparatively less mobile than haul trucks. This would appear to suggest that while conveyors are subjected to efficiency measures that account for performance and quality losses, haul truck efficiency is assessed by accounting for losses only up to operating delays. Also the use of conveyors with, for example, a fully mobile in-pit crusher re-defines the definition of fixed plant. The reason for this exclusion may lie in the difficulty of measuring quality and performance losses associated with trucks since they consist of many units as opposed to conveyors that might have only a few flights or units.
Overall equipment effectiveness

One of the most commonly used measures of equipment efficiency is the Overall Equipment Effectiveness (or Efficiency) – OEE (Emery, 1998 and Mohammadi et al, 2015). OEE is defined by equation 1.

\[
\text{OEE} = \text{Availability} \times \text{Performance} \times \text{Quality}
\]  

(1)

where:
Availability = Operating Time / Planned Production Time;
Performance = (Output / Operating Time) / (Rated Output / Design Cycle Time);
Quality = Valuable Output / Total Output.

Equation (1) can be used for basic estimates of OEE without collecting all six loss categories. A review of literature indicates that further modifications of the OEE definition have evolved since Equation (1) first became commonly applied. Without discussing the details involved, a summary of various modifications to the OEE suggested by Pintelon and Muchiri, (2006) are listed as follows:

- Total Equipment Effectiveness Performance (TEEP)
- Production Equipment Effectiveness (PEE)
- Overall Factory Effectiveness (OFE)
- Overall Asset Effectiveness (OAE)
- Overall Production Effectiveness (OPE)

Pintelon and Muchiri (2006) advanced their argument by proposing a framework that treats the OEE at three levels:
equipment level;
operational level; and
business level.

It is, however, the view of the authors that these approaches over-emphasise the losses rather than focusing on the Input – Output – Loss relationship. Furthermore, this approach fails to account for other key production factors like labour and energy requirement which is a function of the valuable operating time.

MINE PRODUCTION INDEX

Another suggested measure for equipment performance is the Mine Productivity Index (MPI). The proponents argue that the OEE including utilisation has limited application for mining in that, the quality rate cannot be used for the mining industry as per the original definition intended in the OEE equation (Lee and Johnson 2015 and Lanke et al., 2014). They defined MPI as follows:

\[
\text{MPI} = \text{Av}^a \times \text{PP}^b \times \text{U}^c
\]

(2)

Where:
Av = Availability,
PP = Performance and
U = Utilization and 0<a, b, c<=1 and Σ a, b, c=1.

It can be seen that equation (2) essentially replaces Quality with Utilisation and applies a weighting to the various elements through the constants a, b, and c. This may answer the need to use a multi-dimensional measure to assess the overall efficiency of mining equipment.

DATA COLLECTION AND ANALYSIS

Table 1 provides a summary of the order of priority applied to the data collecting and analysis in this paper. The prioritization sought to minimise variations in material properties, data collection methodology, time usage definition and also potential effect of different maintenance practices. Where
unavoidable, data from different operations and time overlaps have been used. After the data was collated, a series of checks were conducted to establish the context of the data and then the data was reorganized, filtered and re-checked. This paper focuses on the measurement of efficiency at the fleet level using the three-fold metric previously discussed.

Table 1: Prioritisation of criteria for data collection and analysis

<table>
<thead>
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<th>Priority</th>
<th>Applied To</th>
<th>Criteria</th>
<th>Data Analysed</th>
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<tbody>
<tr>
<td>1</td>
<td>All</td>
<td>Data from same site / deposit</td>
<td>3 Mining Operations</td>
</tr>
<tr>
<td>2</td>
<td>Trucks</td>
<td>Data from same mine operator/source</td>
<td>5 Rope shovels (44.5m³)</td>
</tr>
<tr>
<td>3</td>
<td>Conveyors</td>
<td>Data from mining application, e.g. IPCC</td>
<td>198 trucks (225t – 360t)</td>
</tr>
<tr>
<td>4</td>
<td>Shovels</td>
<td>Data from Same Type/ Make/Class</td>
<td>309, 000 Load Record</td>
</tr>
<tr>
<td>4</td>
<td>All</td>
<td>All other available sources</td>
<td></td>
</tr>
</tbody>
</table>

A total of 308,912 load records were analysed from three mines operating within Australia with a total of five electric rope shovels and nine different fleets of haul trucks grouped as Type 1 (≤240 tonnes) and Type 2 (>240 tonnes). In total data from 198 trucks have been analysed and the results form the basis for discussion in this paper. The data collected covers a period of at least twelve months with an average of 29 days per month. Due to confidentiality issues the identities of the mines and equipment cannot be revealed. However, the data set used in this analysis is representative of similar class of operating equipment in industry.

LIMITATIONS OF DATA ANALYSIS

It is widely understood that the environment within which mining equipment operates has a significant impact on the performance of the equipment. It is recognised that comparing equipment data from different mining operations without recourse to explaining the unique operating or mine-specific conditions (e.g. different haul profiles, operating bench heights, material characteristics and strip ratios) that might have a bearing on the performance of the equipment could significantly affect the results discussed in this paper. Another issue observed from the data used is the fact that different computer systems or Fleet Management Systems (FMS) and data classification logic might have been used in gathering the data and therefore could also affect the results presented in this paper. Furthermore, the above issue may have been compounded by the lack of full understanding of the context under which the data was collected, e.g. weather issues and operational constraints. An issue identified was underlying error in the data which is difficult to explain without understanding the context of datasets and its impact on the data quality. Notwithstanding, an endeavour has been made to use acceptable statistical techniques and data filtering methods to ensure that the results are reflective of reality.

SHOVEL PERFORMANCE

While the word shovels in this paper refers to any of the following loading equipment types, there is an apparent emphasis on the electric rope shovel as the data collected relates to this category:

Electric Rope Shovels (ERS)
Hydraulic Face Shovels (FS)
Hydraulic Backhoe Excavators (BH)

The following discussions give a brief summary of some of the key issues at the heart of loading equipment performance. Figure 3 shows the working ranges of three examples of some of the largest shovels for digging and loading at the dig face. It is noted that the image is only for enhancing the discussions in this article and therefore does not have specific alignment with any particular OEM.
It can be observed from Figure 3 that all three classes of loading units have nearly the same working range in terms of height but have distinctly different reach and digging depth as a result of equipment sizing in general and front-end arrangement of the buckets, arms and digging mechanism. Table 2 provides a summary of some of the loader-specific characteristics that affect measuring mining equipment performance. These factors not only affect equipment erection costs, but also the operational efficiency in terms of bucket cycle times and maximum bench height used for mining and the capital efficiency. Table 3 shows indicative cycle times and reference Bench Heights (BH) for the three classes of shovels discussed previously. The indicative values show that for the same digging and loading conditions, the BH typically has the least cycle time per pass although bench height may impact these ranges significantly.

Table 2: Prioritisation of criteria for data collection and analysis

<table>
<thead>
<tr>
<th>Factor</th>
<th>BH/FS</th>
<th>ERS</th>
<th>Area of Impact</th>
</tr>
</thead>
<tbody>
<tr>
<td>Machine Assembly Time</td>
<td>Typically takes 10-20 days, with an eight-person crew working a 10-hour shift</td>
<td>It takes approximately 30-70 days to erect an ERS</td>
<td>Capital Productivity</td>
</tr>
<tr>
<td>Mobility</td>
<td>Higher manoeuvrability and mobility shown in faster travel speeds.</td>
<td>Lower manoeuvrability and mobility shown in slower travel speeds.</td>
<td>Operational Efficiency</td>
</tr>
<tr>
<td>Bench Height</td>
<td>Better suited for low – med-height benches. Able to dig below grade to depths up to 8m (BH)</td>
<td>Better suited for high benches. Limited ability to dig below grade.</td>
<td>Operational Efficiency</td>
</tr>
</tbody>
</table>

Table 3: Indicative cycle times and reference bench heights (Fiscor 2007 and Berkhimer 2012)

<table>
<thead>
<tr>
<th>Cycle Times</th>
<th>Units</th>
<th>Rope Shovel</th>
<th>Hyd. Excavator</th>
<th>Face Shovel</th>
<th>Wheel Loader</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dump</td>
<td>s</td>
<td>3.0</td>
<td>3.0</td>
<td>4.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Swing Empty</td>
<td>s</td>
<td>8.3</td>
<td>7.5</td>
<td>6.0</td>
<td>9.0</td>
</tr>
<tr>
<td>Time in Bank</td>
<td>s</td>
<td>11.0</td>
<td>12</td>
<td>13</td>
<td>14.0</td>
</tr>
<tr>
<td>Swing Loaded</td>
<td>s</td>
<td>11.5</td>
<td>7.0</td>
<td>8.0</td>
<td>11.0</td>
</tr>
<tr>
<td>1st Bucket Delay</td>
<td>s</td>
<td>4.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Cycle Time Per Pass</td>
<td>s</td>
<td>37.8</td>
<td>32.5</td>
<td>34.0</td>
<td>43.0</td>
</tr>
</tbody>
</table>
The difference in the cycle time for the ERS lies in the swing times as shown in Table 3. In the case of the FS, the difference in cycle time is due to the time in bank (dig cycle). These differences are supported by the argument that the FS is able to produce higher digging forces low in the bank but ends up with a relatively lower breakout force compared to the ERS. Conversely the ERS consistently generates higher digging forces throughout the bank aided by the hoist force from the ropes as shown in Figure 4 (P&H 2003).

![Figure 4: (a) Digging forces in bank (b) Net digging force profile for shovels (P&H, 2003)](image)

It has been argued that 79% of the shovel's time is spent digging, with the remaining time spent waiting or idling (9%); clean-up work (5%), relocating to new digging position (3.1%), and 3.5% of the time is spent offline (Fiscor 2007). Three main loading approaches are used industry wide. These are double sided loading, single sided loading and single drive-by loading; a modification of the latter is often counted as the fourth method. Truck spotting and operator skill levels play a major role in how efficiently these loading methods are executed (Choudhary 2015). Figure 5 shows the spotting tolerance for the major mining shovels. The ERS has a relatively wider spotting tolerance due to the variable reach of the dipper arm as shown in Figure 5.

![Figure 5: Truck spotting tolerance for the major mining shovels (P&H 2003)](image)

While there are a number of arguments for double-sided loading aimed at reducing truck spotting time, counterarguments suggest that it does not improve the truck cycle time other than eliminating some of the truck queuing, thereby improving the truck exchange time (Bradley 2000). Figure 6 shows an observed trend of spotting time versus loading unit productivity. It shows that an increase of 20 seconds in spotting time could lead to a 500t/hr drop in the shovel productivity (Tegtmeier 2007). A critical review of literature suggests that shovel productivity is significantly affected by the truck spotting time (Fiscor 2007). While there have been a number of efforts to improve the spotting time by using the double sided loading method, it has been argued that double sided loading may marginally reduce the spotting time.
Truck and shovel performance is a function of the Match Factor (MF: ratio of loader productivity rate to truck productivity). Since the MF is essentially the ratio of loader service time to truck arrival rate, much of the effort in this system is focused on balancing the MF. An MF less than 1.0 is indicative of an underproductive or inadequately sized shovel whereas a MF greater than 1 suggests an inadequate truck fleet. The MF is therefore used as a measure to determine the correct truck fleet size (Burt et al., 2005).

Figure 6: Truck spotting tolerance for the major mining shovels (Tegtmeier, 2007)

Shovel performance – time usage

One of the key measures of shovel productivity is the annual Total Material Movement (TMM), which is a function of the average payload per shovel cycle and the number of load cycle per year. The TMM is also a function of the total Operating Time of the equipment. Figure 7 shows the total annual production generated by the five dig units discussed in this paper. Figure 7 shows that the 5 loading units across the three mines are on the average producing 13.5Mbcm with a maximum demonstrated output of 17.5Mbcm per annum. It is also evident from Figure 7 that, while Mine A is achieving average output, Mine B and C are well over the average output with Mine C emerging top with the highest production of 17.5 Mbcm per annum. Figure 8 shows a gap and opportunity result for the variance in worst case and best case production. The results indicate that changes or improvement in bucket fill factor (4%), operator efficiency (3%) and material characteristics (14%, including fragmentation quality) could significantly improve the output of the worst performing shovel. Poor bucket fill factor results in high variability in payload and also a reflection of the operator skill level. The quality of material fragmentation, which is influenced by drilling and blasting practices, has a significant impact on the bucket fill factor. Details of these operational analyses are not discussed further as they fall out of the scope for this paper.

Breakdown of utilised time

Figure 9 shows the breakdown of time usage for the five shovels including availability, operating time, operating delays and operating standby as defined earlier in Figure 2. The upper shading (Grey) is a combination of operating standby and operating delays; added to the OT give the utilised time. Figure 9 shows that the average utilised hours across the 5 shovels is around 6,457 hours as expected for this class of shovel. It is also shown in the figure that losses due to delays and standbys amounts to a third of the utilised time leaving an average OT of 4,514 hours across the 5 shovels. The results also show that
except for SH04 (availability of 78%), all the other shovels achieve an average of 87% availability which is an acceptable. The next sections look at a further breakdown of the OT into loading, spotting and non-loading tasks.
Breakdown of shovel operating time

Figure 10 shows the proportion of truck spotting (plus exchange) to load lime where between 40% and 50% of shovels’ Operating Time is spent spotting trucks (including truck exchange). At first glance, the results are surprising, however a closer look at Figure 11 showing the spotting time of trucks provides further weight to the analysis. Another premise for these results is that the loading method used is a 50% share of single and double sided loading method. Single side loading typically has higher truck queues and spot times compared to the double sided loading method. The key opportunity identified here is that, the use of a continuous mining system like the FM-IPCC may be a good alternative for reducing the shovel’s time lost due to spotting trucks and non-loading tasks within the OT.

A critical review of literature suggests that shovel productivity is significantly affected by the truck spotting time. While there has been a number of efforts to improve the spotting time by using the double-side loading method, it has been advanced that double sided loading may marginally improve the spotting time but has no significant impact on the cycle time of the truck. Figure 11 shows the spotting time of the various classes of truck fleet under the five loading shovels. A clear distinction is observed for Type 1 and Type 2 truck fleets in terms of face loading profiles (queue, spot and loading) of the truck-shovel load interaction.

Definition of truck queue, spot and loading times at the loading face

The truck queue, spot and loading time are defined with respect to a virtual perimeter (beacon) established around the shovel summarised as follows: Queue Time is when the truck has arrived within shovel beacon, <10km/h and another truck loading; Spot Time is the time from last truck full to first bucket or time from arrive to first bucket if no other trucks loading; Loading Time is the time from the first bucket trigger to truck full. Once the truck enters the beacon radius, its locations picked up by GPS and it attains arrive status. The definition of spot time therefore encompasses the combined time from the start of last ‘Full Truck” to the time of next “Load Start”. Spotting therefore includes truck exchange time, first bucket delay and truck positioning. Upon dumping the first bucket, the operator triggers a loading button and attains the “loading state”. Verbal advice from industry experts indicates that the time for a single shovel pass is a fair estimate of truck spot time.

![Figure 10: Proportion of Truck Spotting (plus exchange) to Load Time](image-url)
Figure 11: Breakdown of truck-shovel interaction time for five mining shovels

It is apparent from Figure 11 resulting from the 308,912 load records analysed that, the average truck queue time per load is approximately three times the spot and load times. Spot and load times appear to be 5-15% of the other. It is important to note that the load times (and cycle times) as shown in Figure 11, observed from the results are very good compared to indicative values discussed in Table 4. Spot time as used in this paper refers to the time from last truck full to first bucket which often has an overlap with the definition for truck exchange time. This explains why the spot time of the shovel seems high. There is often a lag between when truck spotting ends and when the truck operator actually triggers the status change. In such instance, the spot times may appear to be longer that they actually are. Figure 12 shows the overall shovel performance the truck fleet which align with observations made in Figure 7, through to Figure 11.

Table 4: Summary of average queue, spot and load times of truck fleet by class

<table>
<thead>
<tr>
<th>Average Of Queue, Spot &amp; Load Time</th>
<th>Type 1 (≤240t)</th>
<th>Type 2 (&gt;240t)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Queue Time</strong></td>
<td>2.94</td>
<td>3.61</td>
</tr>
<tr>
<td><strong>Spot Time</strong></td>
<td>0.86</td>
<td>1.16</td>
</tr>
<tr>
<td><strong>Load Time</strong></td>
<td>0.63</td>
<td>1.31</td>
</tr>
</tbody>
</table>
The primary observation here is that the actual shovel output equates to only a fraction (55%–60%) of the shovel OT which is a maximum of 17.5 Mbcm per annum as shown in these results. This implies that the shovel could produce an additional 20% - 25% which is equivalent to 4.37 – 4.80 Mbcm per annum based on the results discussed earlier in this paper. These figures leave a 15% - 20% leave a 10% room for other non-loading tasks which is still questionable for best practice performance. From the foregoing discussions, the follow conclusions are reached with regard to the shovel productivity efficiency:

- Between half (40% - 50%) of the shovels OT is spent spotting trucks, which shows the inherent losses associated with the Truck-Shovel mining combination.
- The use of an efficient continuous haulage system may improve the productivity of the shovel (Loading units in general) by up to 20% - 25% of current levels. It is acknowledged that the alternative use of FM-IPCC may not totally eliminate shovel production losses associated with truck spotting as the shovel-mobile crusher interaction may incur some production losses.
- The potential benefits accruable from efforts put into the above two opportunities far outweighs any benefits from efforts that may focus on improving the shovels operating hours or utilised hours particularly if the loading unit is already achieving between 4,500 – 5,200 hours OT (excludes operating delays and operating standbys).
- The average VOT of the five shovel across the three mines is 2,483 hours (55% X 4,514 hours) which essentially is the time when the shovel is doing “useful work”.

**TRUCK AND COVEYOR PERFORMANCE**

Table 5 provides a summary of the time utilisation of the 198 truck units analysed over a 12 month period grouped by mine. The results show that the weighted average utilised time per truck is 5,300 hours with a range of 5,200 – 5,800 hours, which again is a fairly good achievement. Figures 13 and Figure 14 show a breakdown of the total truck fleet utilised time and OT respectively.

Table 5: Prioritisation of criteria for data collection and analysis

<table>
<thead>
<tr>
<th>Truck Fleet Size</th>
<th>Util. Time (hours)</th>
<th>Op. Time (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mine A</td>
<td>84</td>
<td>4,893</td>
</tr>
<tr>
<td>Mine B</td>
<td>44</td>
<td>5,294</td>
</tr>
<tr>
<td>Mine C</td>
<td>70</td>
<td>5,844</td>
</tr>
<tr>
<td>Weighted Avg.</td>
<td>198</td>
<td>5,318</td>
</tr>
</tbody>
</table>

Figure 13: Breakdown of OT for the all truck fleet
The focus of this paper is on the effectiveness of performing the objective(s) for acquiring the equipment and therefore the emphasis on truck performance will be on its OT. It can be seen from Figure 13 that 38% of the truck fleet's OT is actually spent travelling empty - often described a carrying dead weight. From these results it can be estimated that the VOT of the entire truck fleet is 2,638 hours (Travelling Full + Spot Time + Loading Time + Dumping Time).

Arguments against this approach could be that since trucks first need to travel empty in order to get loaded, travelling empty must be counted as useful work. Conversely it can be argued that since the extent of dead weight carrying is mainly a function of the haul profile (Grade, rolling resistance and distance), travelling empty should be excluded from useful work particularly when the inbound travel distance differs significantly from the outbound haul distance. Figure 14 shows a breakdown of (a) UT and (b) OT for Conveyors as used in an IPCC configuration. It can be seen from the results presented Figure 14 (a) that the conveyors achieve an average of 4,639 hours in utilised time compared to the average truck utilised time of 5,318 hours (15% higher). The results in Figure 14 (a) also indicate that the conveyors achieve an average of 4,287 hours in Operating Time compared to the average truck Operating Time of 4,254 Hours.

Looking at the Figure 14 (b) it becomes apparent that the conveyors achieve an average of 3,509 Hours in VOT relative to the average truck's VOT of 2,638 hours (25% lower). Table 6 provides a summary of results discussed so far including the time usage of shovels. It is noted that no SMU factors have been applied to these equipment hours.

![Figure 14: Breakdown of (a) Utilised Time (UT) and (b) Operating Time (OT) of conveyor system](image)

<table>
<thead>
<tr>
<th>Equipment Type</th>
<th>UT (hours)</th>
<th>OT (hours)</th>
<th>VOT (hours)</th>
<th>Output Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shovel</td>
<td>6,457</td>
<td>4,514</td>
<td>2,483</td>
<td>28.7 Loads/hour</td>
</tr>
<tr>
<td>Haul Trucks</td>
<td>5,318</td>
<td>4,254</td>
<td>2,638</td>
<td>1.9 Loads/hour</td>
</tr>
<tr>
<td>Conveyors</td>
<td>4,639</td>
<td>4,287</td>
<td>3,509</td>
<td>6,000 t/hour*</td>
</tr>
</tbody>
</table>

*Already crushed material measured in tonnes per hour.

**CONCLUSIONS**

The aim of this paper was to re-evaluate the performance of shovels, trucks and conveyor three key performance measures of: UT, OT and VOT. From the foregoing discussions, the following conclusions are reached with regards to the shovel, truck and conveyor performance:
1. Approximately half (45%) of the shovels OT is spent spotting trucks, which shows the inherent losses associated with Truck-Shovel mining arrangements. The average VOT of the shovel across the three mines was 2,483 hours (50% of the OT).

2. The potential benefits of improving Valuable Operating Time outweigh the benefits from efforts that improve shovel operating hours or utilised hours. Loading units already achieve between 4,500 – 5,200 Hours of Operating Time (excluding operating delays and operating standbys). The use of an efficient continuous haulage system such as an in-pit crusher and conveyor system has the potential to improve the productivity of the shovel (Loading units in general) by between 20%-25% of current levels.

3. While the truck fleet achieve much higher available time and utilisation time, the conveyors achieved higher Operating Time and Valuable Operating Time compared to the truck fleet. The conveyors achieve an average of 3,509 hours in Valuable Operating Time relative to the average truck’s Valuable Operating Time of 2,638 hours (25% lower). These estimates affect the calculation of the capital efficiency of IPCC systems. It is noted that no SMU factors have been applied to these equipment hours.

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Reducing Fuel Consumption of Haul Trucks in Surface Mines Using Artificial Intelligence Models

Ali Soofastaei¹, Saiied Mostafa Aminossadati¹, Mehmet Siddik Kizil¹ and Peter Knights¹

Abstract: Energy saving has become an important aspect of every business activity as it is important in terms of cost savings and greenhouse gas emission reduction. This study aims to develop a comprehensive artificial intelligence model for reducing energy consumption in the mining industry. Many parameters influence the fuel consumption of surface mining haul trucks. This includes, but not limited to, truck load, truck speed and total haul road resistance. In this study, a fitness function for the haul truck fuel consumption based on these parameters is generated using an Artificial Neural Network (ANN). This function is utilised to generate a multi-objective model based on Genetic Algorithm (GA). This model is used to estimate the optimum values of the haulage parameters to reduce fuel consumption. The developed model is generated and tested using real data collected from four large surface mines. It is found that for all four mines considered in this study, the haul truck fuel consumption can be reduced by optimising truck load, truck speed and total haul road resistance using the developed artificial intelligence model.

INTRODUCTION

Energy efficiency has become more important worldwide due to the rise of the cost of fuel in recent years. The Mining industry consumed 450 PJ of energy in 2013-14 or 11% of the national energy use in Australia (BREE 2014)². Mining operations use energy in a variety of ways, including excavation, material transfer, ventilation, dewatering, crashing and grinding operations (DOE 2012)³. Based on completed industrial projects, significant opportunities exist within the mining industry to reduce energy consumption. The potential for energy savings has motivated both the mining industry and governments to conduct research into the reduction of energy consumption (DOE 2002). In addition, a large amount of energy can be saved by improving mining technologies and energy management systems (Kumar Narayan et al., 2010 and Abdelaziz et al., 2011). Energy saving has also a significant positive impact on greenhouse gas emission reduction because the major energy sources used in the mining industry are petroleum products, electricity, coal and natural gas (Asafu and Mahadevan 2003 and Broom 2013).

In surface mines, the most commonly used means of mining and hauling of materials is via a truck and shovel operation (Beatty and Arthur 1989 and Beckman 2012). The trucks used in the haulage operations of surface mines use a great amount of energy (Sahoo et al., 2010 and DOE 2012) and this has encouraged truck manufacturers and major mining corporations to carry out a large number of research projects on the energy efficiency of haul trucks (Chingooshi et al., 2010).

The rate of energy consumption is a function of a number of parameters. The study conducted by Antoung and Hachibli (2007) was concerned with the implementation of power-saving technology to improve the motor efficiency of mining equipment. The focus of their study was on the technical performance of motor components and how they contributed to the reduction of friction and the improvement of the motor efficiency. Beatty and Arthur (1989) investigated the effect of some general parameters, such as cycle time and mine planning, on the energy used by haul trucks. They determined the optimum values of these parameters to minimise fuel consumption in hauling operations. The research presented by Carmichael et al., (2014) was concerned with the effects of haul truck fuel

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³ Department Of Energy (DOE), USA,
consumption on costs and gas emissions in surface mining operations; however, the simulation used in their research did not include the pertinent factors affecting the fuel consumption. Coyle (2007) also researched the effects of load on truck fuel consumption. In this study, Coyle analysed the effect of load density variation based on the blasting procedures on fuel consumption by haul trucks.

The studies reported in the literature are mainly based on the theoretical models used to calculate the fuel consumption of haul trucks. These models are based on the Rimpull-Speed-Grad curve prepared by the truck manufacturer for the performance of trucks (Alarie and Gamache 2002; Beckman 2012; Caterpillar 2013). In this study, the effects of three major parameters; Load (L), Truck Speed (S) and Total Haul Road Resistance (TR), on fuel consumption of haul trucks have been examined. Calculating the impact of these parameters on fuel consumption for a haul truck operating on a real mine site is not an easy task. Therefore, Artificial Neural Network (ANN) and Genetic Algorithm (GA) techniques have been used to develop a model to estimate and reduce fuel consumption. This model has been completed and tested based on comprehensive datasets collected from four large surface mines in The United States and Australia. The developed model can estimate the fuel consumption of haul trucks in surface mines using an ANN and can also find the optimum value of L, S and TR using a GA.

**THEORETICAL CALCULATION OF HAUL TRUCK FUEL CONSUMPTION**

Haul truck fuel consumption is a function of a variety of parameters. Figure 1 shows a schematic diagram of a typical haul truck and the key factors affecting the performance of the truck.

This study examined the effects of the L, S and TR on the fuel consumption of haul trucks. The TR is equal to the sum of the Rolling Resistance (RR) and the Grade Resistance (GR) (Burt et al., 2012).

\[ TR = RR + GR \]  

(1)

The RR depends on tyre and haul road surface characteristics and is used to calculate the Rimpull Force (RF), which is the force that resists motion as the truck tyre rolls on the haul road. The GR is the slope of the haul road, and is measured as a percentage and calculated as the ratio between the rise of the road and the horizontal length (EEO 2012).

The truck Fuel Consumption (FC) can be calculated from Equation 2 (Filas 2002):

\[ *Gross\ Vehicle\ Weight (GVW) = Load + Truck\ Weight \]

\[ Energy\ Efficiency\ Opportunities\ (EEO),\ Australia \]
FC = \frac{SFC}{FD} (LF.P) \quad (2)

Where SFC is the engine Specific Fuel Consumption at full power (0.213–0.268 kg/kw.hr) and FD is the Fuel Density (0.85 kg/L for diesel). The simplified version of Equation 2 is presented by Runge (1998):

FC = 0.3 (LF.P) \quad (3)

Where LF is the engine Load Factor and is defined as the ratio of average load to the maximum load in an operating cycle (Kecojevic and Komljenovic 2010). The typical values of LF are presented in Table 1 (Caterpillar 2013).

<table>
<thead>
<tr>
<th>Operating Conditions</th>
<th>LF (%)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>20 - 30</td>
<td>Continuous operation at an average GVW less than recommended, No overloading</td>
</tr>
<tr>
<td>Medium</td>
<td>30 - 40</td>
<td>Continuous operation at an average GVW recommended, Minimal overloading</td>
</tr>
<tr>
<td>High</td>
<td>40 - 50</td>
<td>Continuous operation at or above the maximum recommended GVW</td>
</tr>
</tbody>
</table>

P is the truck power (kW) and it is determined by:

P = \frac{1}{3.6} (RF.S) \quad (4)

Where the RF is calculated by the product of Rimpull (R) and the gravitational acceleration (g) and S is Truck Speed.

**ESTIMATION OF HAUL TRUCK FUEL CONSUMPTION**

**Artificial Neural Network Model**

Artificial Neural Networks (ANNs) are a popular artificial intelligence model to simulate the effect of multiple variables on one major parameter by a fitness function. ANNs, also known as Neural Networks (NNs), Simulated Neural Networks (SNNs) or ‘parallel distributed processing’, are the representation of methods that the brain uses for learning (Picton 1994). ANNs are a series of mathematical models that imitate a few of the known characteristics of natural nerve systems and sketch on the analogies of adaptive natural learning. The key component of a particular ANN paradigm could be the unusual structure of the data processing system. ANNs are utilised in various computer applications to solve complex problems. They are fault-tolerant and straightforward models that do not require information to identify the related factors (LeCun et al., 1998) and do not require the mathematical description of the phenomena involved in the process. This method can be used to determine fuel consumption by taking into consideration a number of variables that influence the fuel consumption of haul trucks. ANNs have been used in many engineering disciplines such as materials (Sha and Edwards 2007; Reihanian et al., 2011; Hammood 2012; Pourasiabi et al., 2012 and Xiang et al., 2014), biochemical engineering (Talib et al., 2009), medicine (McCulloch and Pitts 1943) and mechanical engineering (Ekici and Aksoy 2009; Rodriguez et al., 2013 and Beigmoradi et al., 2014).
In this study, an ANN was developed to create a Fuel Consumption Index (FC\textsubscript{Index}) as a function of L, S and TR. This index shows how many litres of diesel fuel are consumed to haul one tonne of mined material in one hour.

**DEVELOPED MODEL**

The main part of a neural network structure is a 'node'. Biological nodes generally sum the signals received from numerous sources in different ways and then carry out a nonlinear action on the results to create the outputs. Neural networks typically have an input layer, one or more hidden layers and an output layer. Each input is multiplied by its connected weight and in the simplest state, these quantities and biases are combined; they then pass through the activation functions to create the output (see Equations 5, 6, 7). Figure 2 shows a simple structure of developed model in this study (it should be noted that the hidden layer nodes may use any differentiable activation function to generate their output).

\[
E_k = \sum_{j=1}^{q} (w_{i,j,k} x_j + b_{i,k}), \quad k = 1, 2, ..., m
\]  

(5)

Where \(x\) is the normalised input variable, \(w\) is the weight of that variable, \(i\) is the input, \(b\) is the bias, \(q\) is the number of input variables, and \(k\) and \(m\) are the counter and number of neural network nodes, respectively, in the hidden layer.

**Figure 2: A simple structure of ANN developed model**

In general, the activation functions consist of both linear and nonlinear equations. The coefficients associated with the hidden layer are grouped into matrices \(W_{i,j,k}\) and \(b_{i,k}\). Equation 6 can be used as the activation function between the hidden and the output layers (in this equation, \(f\) is the transfer function).

\[
F_k = f(E_k)
\]  

(6)
The output layer computes the weighted sum of the signals provided by the hidden layer and the associated coefficients are grouped into matrices $W_{o,k}$ and $b_o$. Using the matrix notation, the network output can be given by Equation 7.

$$\text{Out} = \left( \sum_{k=1}^{m} w_{o,k} f_k \right) + b_o$$  \hspace{1cm} (7)

This paper presents a study in which different types of algorithms were examined in order to determine the best back-propagation generating algorithm. In comparison to other back-propagation algorithms, the Levenberg–Marquardt (LM) back-propagation generating algorithm has the minimum Mean Square Error (MSE), Root Mean Square Error (RMSE) and Correlation Coefficient ($R^2$) (See Equations 8, 9 and 10).

In addition, network generating with the LM algorithm can run smoothly with the minimum Expanded Memory Specification (EMS) and a fast generating process. MSE, RMSE and $R^2$ are the statistical criteria utilised to evaluate the accuracy of the results according to following equations (Ohdar and Pasha 2003 and Poshal and Ganesan 2008):

$$\text{MSE} = \frac{1}{p} \sum_{r=1}^{p} (y_r - z_r)^2$$  \hspace{1cm} (8)

$$\text{RMSE} = \left( \frac{1}{p} \sum_{r=1}^{p} (y_r - z_r)^2 \right)^{\frac{1}{2}}$$  \hspace{1cm} (9)

$$R^2 = 1 - \frac{\sum_{r=1}^{p} (y_r - z_r)^2}{\sum_{r=1}^{p} (y_r - \bar{y})^2}$$  \hspace{1cm} (10)

Where $y$ is the target (real), $z$ is the output (estimated) of the model, $\bar{y}$ is the average value of the targets and $p$ is the number of the network outputs (Demuth and Beale 1993 and Krose et al., 1993). In this study, the MSE and $R^2$ methods were applied to examine the error and performance of the neural network output and the LM optimisation algorithm was utilised to obtain the optimum weights of the network.

**NETWORK RESULTS**

Figures 3, 4, 5 and 6 illustrate the correlation between $P$, $S$, $TR$ and $FC_{\text{Index}}$ created by the developed ANN model for a normal range of loads for four types of popular trucks used in four big surface mines in The United States and Australia. The presented graphs show that there is a nonlinear relationship between $FC_{\text{Index}}$ and $P$. The rate of fuel consumption increases dramatically with increasing TR. However, this rate does not change sharply with changing truck speed, $S$. 
Figure 3: Correlation between L, S, TR and $FC_{\text{index}}$ based on the developed ANN model for CAT 793D. All data have been collected from a surface coal mine located in the Central Queensland, Australia (Mine 1).

Figure 4: Correlation between GVW, S, TR and $FC_{\text{index}}$ based on the developed ANN model for CAT 777D. All data have been collected from a surface copper mine located in Arizona, USA (Mine 2).
Figure 5: Correlation between GVW, S, TR and $F_{C_{\text{index}}}$ based on the developed ANN model for CAT 775G. All data have been collected from a surface copper mine located in Arizona, USA (Mine 3)

Figure 6: Correlation between GVW, S, TR and $F_{C_{\text{index}}}$ based on the developed ANN model for CAT 793D. All data have been collected from a surface coal mine located in Arizona, USA (Mine 4)

**ANN GENERATING AND VALIDATION**

In order to generate the proposed ANN model, 1,000,000 data were randomly selected from the collected real datasets from four mine sites individually. In order to test the network accuracy and validate the model, 1,000,000 independent samples were used again. The results show good agreement between the actual and estimated values of fuel consumption. Figures 7 presents sample
values for the estimated (using the ANN) and the independent (tested) fuel consumption in order to highlight the insignificance of the values of the absolute errors in the analysis for four studied mines.

![Graph showing estimated vs. independent fuel consumption values](image)

**Figure 7:** Sample values for the estimated (using the ANN) and the independent (tested) Fuel Consumption Index.

**OPTIMISATION OF EFFECTIVE PARAMETERS ON HAUL TRUCK FUEL CONSUMPTION**

**Genetic Algorithm**

Genetic Algorithms (GAs) were proposed by Holland (1975) as an abstraction of biological evolution, drawing on ideas from natural evolution and genetics for the design and implementation of robust adaptive systems (Amy et al., 2012). The new generation of GAs are comparatively recent optimisation methods. They do not use any information of derivate, therefore, they have a good chance of escape from local minimum. Their application in practical engineering problems generally leads to optimal global solutions, or, at least, to solutions more satisfactory than those ones obtained by other traditional mathematical methods. They use a direct analogy of the evolution phenomena in nature. The individuals are randomly selected from the search area. The fitness of the solutions, which is the result of the variable that is to be optimised, is determined subsequently from the fitness function. The individual that generates the best fitness within the population has the highest chance to return in the next generation, with the opportunity to reproduce by crossover, with another individual, producing decedents with both characteristics. If a genetic algorithm is developed correctly, the population (group of possible solutions) will converge to an optimal solution for the proposed problem. The processes that have more contribution to the evolution are the crossover, based in the selection and reproduction and the mutation (see Table 2 and Figure 8).
Table 2: Genetic Algorithm processes (Goldberg 1989)

<table>
<thead>
<tr>
<th>Process</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initialisation</td>
<td>Generate initial population of candidate solutions</td>
</tr>
<tr>
<td>Encoding</td>
<td>Digitalise initial population value</td>
</tr>
<tr>
<td>Crossover</td>
<td>Combine parts of two or more parental solutions to create new</td>
</tr>
<tr>
<td>Mutation</td>
<td>Divergence operation. It is intended to occasionally break one or more members of a population out of a local minimum space and potentially discover a better answer.</td>
</tr>
<tr>
<td>Decoding</td>
<td>Change the digitalized format of new generation to the original one</td>
</tr>
<tr>
<td>Selection</td>
<td>Select better solutions (individuals) out of worse ones</td>
</tr>
<tr>
<td>Replacement</td>
<td>Replace the individuals with better fitness values as parents</td>
</tr>
</tbody>
</table>

GAs have been applied to a diverse range of scientific, engineering and economic problems (Velez 2005; Opher and Ostfeld 2011; Reihanian et al., 2011; Amy et al., 2012 and Beigmoradi et al., 2014) due to their potential as optimisation techniques for complex functions. There are four major advantages when applying GAs to optimisation problems. Firstly, GAs do not have many mathematical requirements in regard to optimisation problems. Secondly, GAs can handle many types of objective functions and constraints (i.e., linear or nonlinear) defined in discrete, continuous or mixed search spaces. Thirdly, the periodicity of evolution operators makes GAs very effective at performing global searches (in probability). Lastly, The GAs provide a great flexibility to hybridize with domain dependent heuristics to allow an efficient implementation for a specific problem. It is also important to analyse the influence of some parameters in the behaviour and in the performance of the genetic algorithm, to establish them.
according to the problem necessities and the available resources. The influence of each parameter in the algorithm performance depends on the class of problems that is being treated. Thus, the determination of an optimised group of values to these parameters will depend on a great number of experiments and tests.

There are a few main parameters in the GA method. Details of these five key parameters are tabulated in Table 3.

<table>
<thead>
<tr>
<th>GA Parameter</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fitness Function</td>
<td>The main function for optimisation</td>
</tr>
<tr>
<td>Individuals</td>
<td>An individual is any parameter to apply into the fitness function. The value of the fitness function for an individual is its score.</td>
</tr>
<tr>
<td>Populations and Generations</td>
<td>A population is an array of individuals. At each iteration, the GA performs a series of computations on the current population to produce a new population. Each successive population is called a new generation.</td>
</tr>
<tr>
<td>Fitness Value</td>
<td>The fitness value of an individual is the value of the fitness function for that individual.</td>
</tr>
<tr>
<td>Parents and Children</td>
<td>To create the next generation, the GA selects certain individuals in the current population, called parents, and uses them to create individuals in the next generation, called children.</td>
</tr>
</tbody>
</table>

The principal genetic parameters are the size of the population that affects the global performance and the efficiency of the genetic algorithm, the mutation rate that avoids that a given position remains stationary in a value, or that the search becomes essentially random.

**MODEL RESULTS**

In this study, a GA model was developed to improve the key effective parameters on the energy consumption of haul trucks. In this model L, S and TR are the individuals and the main function for optimisation of the fitness function is fuel consumption. In this model a fitness function was created by the ANN Model. In this developed model, the main parameters used to control the algorithm were $R^2$ and MSE. The population size for the first generation was 20 and a uniform creation function was defined to generate a new population. The completed ANN and GA model were developed by writing computer codes in MATLAB software. L, S and TR are inputs of the code in the first step. The completed code creates the fitness function based on the developed ANN model. This function is a correlation between haul truck fuel consumption, L, S and TR. After the first step, the completed function goes to the GA phase of the computer code as an input. The developed code starts all GA processes under stopping criteria defined by the model (MSE and $R^2$). Finally, the improved L, S and TR will be presented by the code. These optimised parameters can be used to minimise the fuel consumption of haul trucks. All processes in the developed model work based on the present dataset collected from four large surface mines, but the completed method can be developed for other surface mines by replacing the data. The results of using developed model for real mentioned mines are tabulated in tables 4 to 7.
### Table 4: The range of normal values and optimised range of variables by GA model to minimise fuel consumption by haul trucks. (Caterpillar 793D in Mine 1)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Normal Values</th>
<th>Optimised Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Gross Vehicle Weight (tonne)</td>
<td>150</td>
<td>380</td>
</tr>
<tr>
<td>Total Resistance (%)</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>Truck Speed (Km/hr)</td>
<td>5</td>
<td>25</td>
</tr>
</tbody>
</table>

### Table 5: The range of normal values and optimised range of variables by GA model to minimise fuel consumption by haul trucks. (Caterpillar 777D in Mine 2)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Normal Values</th>
<th>Optimised Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Gross Vehicle Weight (tonne)</td>
<td>65</td>
<td>150</td>
</tr>
<tr>
<td>Total Resistance (%)</td>
<td>9</td>
<td>25</td>
</tr>
<tr>
<td>Truck Speed (Km/hr)</td>
<td>10</td>
<td>45</td>
</tr>
</tbody>
</table>

### Table 6: The range of normal values and optimised range of variables by GA model to minimise fuel consumption by haul trucks. (Caterpillar 775G in Mine 3)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Normal Values</th>
<th>Optimised Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Gross Vehicle Weight (tonne)</td>
<td>45</td>
<td>85</td>
</tr>
<tr>
<td>Total Resistance (%)</td>
<td>13</td>
<td>20</td>
</tr>
<tr>
<td>Truck Speed (Km/hr)</td>
<td>5</td>
<td>55</td>
</tr>
</tbody>
</table>

### Table 7: The range of normal values and optimised range of variables by GA model to minimise fuel consumption by haul trucks. (Caterpillar 785D in Mine 4)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Normal Values</th>
<th>Optimised Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Gross Vehicle Weight (tonne)</td>
<td>125</td>
<td>215</td>
</tr>
<tr>
<td>Total Resistance (%)</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>Truck Speed (Km/hr)</td>
<td>5</td>
<td>45</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The aim of this study was to develop a model based on the ANN and GA methods to improve haul truck fuel consumption. The relationship between L, S, TR and FC in an actual mine site is complex. In the first part of the study, an ANN method was developed to find a correlation between the key parameters and FC. The results showed that FC has a nonlinear relationship with the investigated parameters. The ANN was generated and tested using the collected real mine site datasets and the results showed that there was good agreement between the actual and estimated values of FC. In the last part of the study, to improve the energy efficiency in haulage operations, a GA method was developed. The results showed that by using this method, optimisation of the effective parameters on energy consumption was possible. The developed method was used to estimate the local minimums for the fitness function. The presented genetic algorithm method highlighted the acceptable results to minimise the rate of fuel consumption. The range of all studied effective parameters on fuel consumption of haul trucks was optimised, and the best values of P, S and TR to minimise FC were highlighted. The developed model was applied to analyse data for four big coal and metal surface mines (Open-Cut and Open-Pit) in the United States and Australia.

ACKNOWLEDGEMENTS

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REFERENCES


IMPROVING PERFORMANCE IN THIN SEAM OPEN CUT MINING – APPLICATION OF PICK BASED CUTTING TECHNOLOGY

Nicole Prochnau¹, Ross Carter² and Hermann Volk³

ABSTRACT: With the current challenges facing the Australian coal industry, a major Australian coal miner has achieved significant improvements from the implementation of a continuous mining system, a Wirtgen Surface Miner 4200 SM, at an open cut coal mine in South East Queensland. Due to the multiple thin seam characteristics of the deposit, selective mining practices are critical. This often results in decreased productivity when using Conventional Mining (CM) equipment. The Wirtgen Surface Miner (WSM) cuts, crushes and loads coal and Interburden (IB) onto a truck in a single step and thereby replaces multiple CM equipment for ripping, stacking and loading. This paper presents an in-field study, assessing the benefits gained from implementing the 4200 SM. When implementing new technology it is important to evaluate its performance compared to the existing system. The trial program was therefore structured around Key Performance Indicators (KPIs) referring to the CM system. The evaluation contains empirical assessments combined with theoretical calculations and literature research. A key challenge of the project was to compare a continuous mining system, i.e. the 4200 SM, against a discontinuous, multi-handling mining system. A commensurable evaluation of both mining systems was achieved by defining a CM-unit. Objectives of the evaluation were productivity, unit costs, Health-Safety and Environmental (HSE) performance, deposit recovery, Run-Of-Mine (ROM) coal quality and impacts on the Coal Handling and Processing Plant (CHPP). The WSM has demonstrated increased productivity, improved HSE performance, minimised loss and dilution, positive impacts on the CHPP, more consistent particle size distribution containing more target product size, as well as significant mining unit cost savings compared to the conventional dozer rip, stack and load process. On average, the mining unit costs are reduced by about 60%, considering different rock properties in coal and IB. The fuel usage per volume mined decreases even more significantly, resulting in a reduction of carbon emissions.

INTRODUCTION

The New Hope Group has recently successfully implemented a Wirtgen Surface Miner (WSM) into the existing mining fleet at its New Acland Coal mine (NAC). The characteristics of the deposit are banded coal plies with thinly bedded sandstone, siltstone and mudstone layers. A total number of 47 plies exist. Coal seam thicknesses vary from less than 0.2 m to about 2 m. This multi thin seam operation requires selective mining practices (Pippenger 2014).

A Wirtgen 4200 SM commenced operation at NAC in June 2014. A six-month trial period was part of the implementation process on site. During that trial period, the performance of the WSM was analysed to evaluate the outcomes against defined Key Performance Indicators (KPIs). Operation of the WSM was trialled in a variety of operating conditions in overburden, Interburden (IB) and coal. The objective of this paper is the analysis of the 4200 SM performance during the first four months of implementation. The project aimed at identifying benefits that the high capacity selective WSM provides to the overall production cycle at NAC. Major expectations related to the WSM mining system were:

- Better coal recovery (less loss and dilution);
- Increased productivity;
- Unit cost savings;

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- Decreased wear and better handling of ROM coal at Coal Handling and Processing Plant (CHPP) (cleaner coal, amount of fines and target particle size);
- Improved handling of seams that contain siderite intrusions;
- Less complex and more efficient mining due to less equipment involved; and
- Positive environmental impacts (e.g. less noise, carbon emission, less blasting).

The trial program was structured around KPIs referring to the Conventional Mining (CM) system. (Wirtgen Australia 2014) During the trial period, the CM process was also examined in detail to provide a basis for a direct comparison of both mining systems. The research project assessed the 4200 SM performance by making a series of comparisons between:

- CM of coal and IB, that utilises a combination of D11 dozers to rip and push the coal / IB and a front-end loader to load trucks; and
- 4200 SM mining of coal and IB, where the 4200 SM mills the coal / IB and directly loads the cut material into trucks.

Both mining systems were assessed regarding environmental, economic and operational parameters. The assessment was not limited to the actual mining process, but also analysed the impacts on the overall process cycle of the mine site.

**WIRTGEN SURFACE MINER TECHNOLOGY**

Amongst various innovative mining technologies, the surface miner technology has gained considerable prevalence over the past decades (Williams et al., 2007). Surface miners are commonly used in North America and Central Asia, in mostly soft rock operations. In Western Australia, a large fleet of WSMs with a hard rock cutting drum is employed in iron ore (Anon 2010). The German machine manufacturer Wirtgen GmbH is currently technology and market leader for surface miners (Berkhimer 2011 and Schimm 1999).

Surface miners perform rock-breakage by means of cutting. A rotating drum with cutting tools (i.e. picks) cuts the material. There are a number of manufacturers that distribute surface miners (Anon 2010). Each manufacturer has its own specific surface miner technology. The main difference is the drum position. Surface miners are available as front drum types, middle drum types and rear drum types. Another difference in the surface miner operating method is the type of material handling. The material is either deposited on the ground or directly loaded onto trucks by an attached conveyor belt.

WSMs look similar to cold milling machines used in road construction and their basic operation principle is the same. They combine cutting, primary crushing and loading in one single step. If direct loading is not desired, windrow machines are available that leave the cut material on the ground between the crawlers. Furthermore, WSMs with a conveyor have the option to side-cast the cut material (Wirtgen GmbH 2010). Wirtgen is the only manufacturer who offers all three material handling options (i.e. direct loading, windrowing, side-casting). The machine set-up can be seen in Figure 1 (Wirtgen GmbH 2010).

![Figure 1: Wirtgen Surface Miner Technology (Wirtgen GmbH 2010)](image-url)
The cutting drum is located between front and rear tracks, in the middle of the machine. This location, close to the centre of gravity, provides efficient transfer of the cutting forces because the whole machine weight and power is transferred into cutting force. Impact loads and shocks are well absorbed and high machine stability is achieved. Toolholders are mounted to the cutting drum in an application-specified layout. The toolholders are fixed to the drum in a helix layout that directly carries the cut material to the centre of the drum and further onto the primary conveyor. The actual cutting tools are round shank bits (i.e. picks) with tungsten-carbide inserts. The design of the cutting system varies depending on mine site conditions (Wirtgen GmbH 2010). The main design factors listed by Wirtgen GmbH (2010) are:

- Cutting tool type;
- Cutting tool holder type;
- Number of cutting tools (spacing);
- Angle of cutting tools; and
- Cutting drum rotation speed.

The 4200 SM implemented at NAC is a soft rock, direct loading machine with a 16 m discharge conveyor belt that has 180° slewing capacity. 130 -180 t payload size trucks are allocated to the WSM at NAC. However, WSMs are able to load 240 t size trucks. A boom counterweight ensures the machine’s stability. The cutting drum has a width of 4200 mm and a diameter of 1860 mm including cutting tools. The drum is equipped with 62 cutting tools in total and rotates upwards. Cutting depths up to 830 mm can be achieved. The maximum operation grade is 20% and lateral inclination is limited to 8% (Wirtgen GmbH 2010). The rated power is 1194 kW (Wirtgen Australia, 2014). The 4200 SM cuts lanes out alongside each other. At the end of each lane, the 4200 SM cuts its own ramp and turns 180° around to cut the next lane. The ramp area and turnaround area is later ripped and stacked into windrows by a dozer and the 4200 SM picks up the windrows before cutting the next bench (Wirtgen GmbH 2010).

CONVENTIONAL MINING SYSTEM

The existing CM system at NAC is a discontinuous process. Rock-breakage and loading occur in separate steps. The main production fleet for coal and IB consists of D11 dozers for rock-breakage and stacking and 992G front-end loaders for excavation. Haulage is carried out by 130 -180 t haul trucks. The CM system is operated in 150 m x 150 m blocks. There is only one mining step (i.e. rock-breakage and stacking or excavation) carried out in one block at a time. Multiple blocks are always held open for mining (Pippenger 2014).

Dozers play a major role at this mine site. They are used for ripping and stacking of coal and IB, which are a major component of the CM system. For loosening the rock for later excavation, the ripper shank penetrates the material and is pulled through the rock. The installed blade stacks the material onto piles for later loading (Smith 1986).

Ripping and dozing performance fluctuates extremely among different materials as well as within the same rock based upon rock mass structure and rock properties. Key aspects of good performance are operator skills and operating techniques. The applied technique should be adjusted to site requirements such as fines generation, dilution and productivity of the loader unit (Doktan and Scott 1998).

Ripping techniques are listed by Humphrey and Wagner (2011) as:

- Cross ripping / Straight ripping;
- Direction of ripping;
- Spacing between ripper passes;
- Direction of pushing (Front-to-back / Back-to-front / Back-each-pass);
- Uphill / Downhill ripping;
- Ripping depth; and
- Angle of shank.
Every mine site has its own ripping techniques associated to the specific site conditions. Ripping techniques affect productivity, dilution and material sizing including fines generation. At NAC the spacing between ripper passes is usually half a track width (SME 1983). The typical shank angle is 15° past vertical. Other criteria such as ripping direction and ripping depth are highly dependent on job requirements and rock mass properties. Another important factor of dozer efficiency is the pushing distance. As a rule of thumb the pushing distance should not exceed 150 m (Humphrey and Wagner 2011). In some conditions it may be required to pre-blast the rock to achieve efficient dozer production (SME 1983) (Caterpillar 1989).

Following rock-breakage, front-end loaders are the equipment of choice for loading the material onto trucks at NAC. Cat 992G front end-loaders equipped with 12 m³ buckets are the subject of this study. The combination – dozer and front-end loader – results in reduced breakout force requirements for the loader unit, because the material is already pre-ripped. Thus, front-end loaders provide good performance and offer increased mobility and flexibility as opposed to bigger excavators. This makes front-end loaders favourable especially in multiple thin seam mining operations (Humphrey and Wagner 2011).

CHALLENGE OF EVALUATION

A challenge lies in comparing a discontinuous CM mining system versus a continuous mining system (i.e. 4200 SM). Reasonable values for a performance evaluation of both mining methods are gathered by reporting upon the complete mining process. It was important to capture data from all mining units allocated to each process. The 4200 SM makes up one WSM-unit, and the dozers plus loaders form the CM-unit. In-the-field data reporting on different levels was the basis of the performance comparison combined with various trial set-ups. A reporting system was developed for analysing production performance of both mining systems. The reporting system resorts to the mine site's internal reporting system. Production rates (IB and coal) are stated in bcm/op.h.

Production rate ranges

For comparing productivity of both mining systems, Production Rates (PR) for the mining units were developed. Data analysis of daily PRs and time studies provided the basis for productivity estimation. Additionally, theoretical PR estimations, based on manufacturer information, were performed. Another resource for PR estimation was historical data provided by the mining company. By comparing the results of the productivity estimation approaches and applying mine site specific assumptions, a considerable PR range for each piece of mining equipment (i.e. D11 dozer, 992G loader, 4200 SM) was established. Multiple parameters, such as rock mass properties, mine design and seam thickness, influence PRs. PR ranges were developed to account for different rock conditions and seam thicknesses. The PR ranges provide a guideline for average productivity.

The established PR range for the 992G loader is 550-750 bcm/op.h in IB and 700-850 bcm/op.h in coal. The D11 dozer PR range for ripping and stacking lies between 300-550 bcm/op.h in IB and 450-600 bcm/op.h in coal. The 4200 SM cuts and loads 700-1050 bcm/op.h in IB and 800-1350 bcm/op.h in coal. The identified PR ranges form the basis for further analysis. All calculations were performed considering the different scenarios (low / high PRs) for coal and IB. Figure 2 shows the average PRs of each type of equipment. The ranges are illustrated as error bars.

Equipment required

The project target PRs of 700 bcm/op.h in IB and 800 bcm/op.h in coal (i.e. the project KPIs), are the basis for analysis. By dividing the target PRs by the applicable PR ranges identified for each machine, the number of necessary pieces of equipment to achieve target PR was identified, according to Equation 1 and Equation 2. The CM system was considered as one unit. Thus, the amount of machines required for rock-breakage and loading was summed up.
Amount of equipment required \( CM = \frac{KPI \text{ Dozer } PR}{KPI \text{ Loader } PR} + \frac{KPI \text{ Loader } PR}{PR} \) (1)

Amount of equipment required \( 4200 \text{ SM} = \frac{KPI \text{ PR}}{4200 \text{ SM PR}} \) (2)

This analysis takes different rock conditions and seam thicknesses into account, represented as low and high PRs. The amount of equipment required to fulfill target production is stated in decimal digits. This allows more detailed comparison of the mining systems capacities. The average machinery requirements are reduced by about 70% when using the 4200 SM in comparison to the CM system, as shown in Figure 3.

![Figure 2: Production rate ranges](image)

![Figure 3: Amount of equipment required](image)

HEALTH - SAFETY AND ENVIRONMENTAL PERFORMANCE

The WSM is a new technology at NAC, which involves new potential safety risks. The conveyor system, cutting technology and vertical edges along the cutting lane were amongst new risks identified in risk assessments. Procedures and controls were developed to manage these potential risks.

The reduction in operating units at the working area and working along predefined cutting lanes decrease the risk of collision, which is considered to be one of the main risks of the CM system. The environmental impact assessment revealed better environmental performance of the WSM compared to the CM system. Reduced machinery usage results in less noise emission, dust generation and carbon emissions. Figure 4 shows the fuel consumption of both mining system. The average fuel usage in l/bcm of the WSM is 68% lower than the combined fuel usage of dozers and loaders in the conventional system. Positive effects were also observed in regards to operators’ ergonomics.
Cost savings are a major driver for implementing new technology. Information provided by the mine site, experience from other WSM applications and trial data formed the basis of unit costs analysis. Dozers and loaders were considered as one mining unit. In Figure 5 the unit costs of both mining systems can be seen. The unit costs of the CM system range from about 2.00 - 4.60 AUD/bcm. The 4200 SM operating costs vary between about 0.70 - 1.40 AUD/bcm. The unit cost ranges take the different PR scenarios based on rock properties into consideration. Ownership, maintenance, labour, plant rate, fuel and GET (e.g. picks, ripper boots, and tracks) are included in the unit cost calculation. Overall, an average reduction of about 60% in unit mining costs is achieved by the WSM system.

Selective mining minimises loss and dilution and thereby improves deposit recovery. At NAC, coal and IB can be well differentiated visually (black-white interface). Dozer operators have good skills in mining the different layers separately.

A camera system is installed on the WSM to provide material detection assistance to the operator. Cameras are located behind the cutting drum on the left and right hand side of the drum, sending a high-resolution image stream of the cut floor to the operator cabin. High resolution and colour rendering screens display the floor in the cabin, as shown in Figure 6. Surrounded by a box and fitted with lights and a ventilation system, the cameras are isolated from external factors like dust and sunlight. Similar viewing conditions during day and night are provided. Together with the cutting depth control, this camera system enables highly selective mining. This material detection system works only on a direct-loading WSM.

Due to good operator skills, loss and dilution are minimal in both mining systems. Visual observations during the trial period confirmed selective mining practice. Surveying data (i.e. actually mined material) and CHPP yields are regularly cross-checked with the estimated volumes from the mine scheduling.
software to identify loss and dilution percentages. This cross-check usually covers a period of 18 months. Since the trial period was limited to four months, this data was only available for the CM system. In lieu thereof, the following theoretical approach was used to estimate loss and dilution of the 4200 SM.

![Figure 6: Material detection camera and screen](image)

While cutting along the interface between IB and coal, the cutting drum fluctuates around the interface. Wirtgen developed a theory that describes this fluctuation. The fluctuation around the interface can be described as a sine-curve with an amplitude of approximately 25 mm (i.e. 1 inch), as shown in Figure 7. Wirtgen states the amplitude of around ± 25 mm based on practical experience. Visual inspections as well as operator and supervisor assessments confirm this assumption. The length of the period is assumed to be about 6 m, meaning that the operator corrects the cutting depth 3 m after recognising he is cutting too deep / shallow (Heinrichs 2014).

![Figure 7 – WSM loss and dilution model (Heinrichs 2014)](image)

Equation 3 describes the sine curve. Based on this sine curve equation and strata information, the percentile recovery can be calculated. This theoretical approach was applied to strata data provided by the mine site. The average loss and dilution are both 3% based on this approach. This indicates better resource recovery and reduced dilution compared to the CM system.

\[ y = A \times \sin(bx + t) \]  
\[ \text{where} \]

A: Amplitude  
b: Period  
t: Phase

**IMPACT ON THE COAL HANDLING AND PROCESSING PLANT**

The ROM coal properties of both mining systems vary in regards to Particle Size Distribution (PSD), particle shape and dilution. Understanding the impacts of the different mining systems on the CHPP is relevant for the evaluation of the mining systems. The quality comparison was performed on the basis of mine site personnel assessments and a detailed PSD trial. Due to the limited trial period, detailed analysis of CHPP yield and final product quality could not be conducted.
A large scale bulk testing, on-site PSD trial was performed at NAC as part of the 4200 SM trial. The purpose of the PSD trial was to compare the PSD of both mining systems in similar conditions. A large-scale trial was considered to achieve more reliable results for mining applications than standard laboratory methods. The trial setup mainly consisted of a Kleemann Screening Unit MS19D and a Sweco Screening Tower for fine material. The Kleemann MS19D is a mobile screening plant, consisting of a grizzly and a triple deck screening unit (Ranft and Klein 2014). The following sizes were separated by the Kleemann screening plant:

- 150 mm;
- 40-150 mm;
- 16-40 mm;
- 4-16 mm; and
- < 4 mm.

The section finer than 4 mm was further analysed by the Sweco screening tower into:

- 2-4 mm;
- 1-2 mm;
- 0.5-1 mm and
- < 0.5 mm (undersize) (Ranft and Klein, 2014).

Sample sizes were approximately 30 – 40 t. Sample collection in the pit from both mining systems occurred according to the usual loading practice. The location, where the sample was taken, material data, and operational data was recorded. Commissioning was carried out prior to the actual PSD trial to confirm accuracy and correctness of sampling and the process (Ranft and Klein 2014).

Trucks were loaded in the pit and dumped at the trial ROM pad. At the trial area, a small front-end loader fed the coal to the screening plant. Each fraction was weighed after the screening process to derive the PSD of each sample. The PSD trial showed that 60% of the conventionally excavated material met CHPP target particle size requirement (i.e. 2 mm – 38 mm), whereas the WSM produces about 70% of this fraction. The WSM produces 10% more CHPP target particle size. WSM coal contains less over-sized coal and requires less crushing. The PSD trial also indicates that the CM system creates more fines compared to the WSM. The absolute reduction in fines generation is 2.3%. The relative reduction in fines generation compared to the CM system is 15.5%. The PSD curves can be seen in Figure 8 (Ranft and Klein 2014).

![Figure 8: Av. PSD, WSM vs. CM (Ranft and Klein 2014)](image)

Besides PSD, the handling of coal that contains siderite intrusions has a major impact on the CHPP. NAC has two CHPPs with different setups. CHPP1 uses a JIG to separate coarse reject, whereas CHPP2 has only a dry circuit and more crushing is required.

Some coal seams contain lenticular siderite intrusions. Those siderite intrusions cause problems in the CHPP when mined with the conventional dozer-rip operation. The rippers often just scratch the surface...
of the siderite or produce oversized material that is unsuitable for the CHPP (+300 mm). Ripping siderite increases wear of the ripper boots and siderite causes bogging and increased wear to the CHPP crushers. Due to the great wear of the CHPP crushers, siderite ROM coal is processed in CHPP1 by preference. However, the high density siderite causes bogging in the JIG and the separation effect decreases. This often results in a breakdown of the CHPP. Therefore, conventional mined siderite coal can only be processed before a planned maintenance shutdown of CHPP1. Consequently, the flexibility to deliver coal of a certain quality at a desired time is limited and scheduling requirements rise.

During the 4200 SM trial period at NAC the performance of the 4200 SM in siderite coal seams was trialled to assess how the 4200 SM handles both – cutting siderite without compromising performance and fragmentation of the siderite. The coal was directly fed to the CHPP. The 4200 SM has proven good cutting performance of both; coal and siderite. There is no significant difference compared to other coal seams. Tool wear remains unchanged. Visual assessment of the ROM and at the CHPP indicates good fragmentation of the siderite. The 4200 SM cuts the siderite down to a particle size smaller than 150 mm, which is more suitable for the CHPPs (McDonald 2014). The 4200 SM siderite coal does not cause any problems at crushers or the JIG. It can be processed at any time at both CHPPs, which increases the flexibility of coal quality management and simplifies mine planning.

CONCLUSIONS

In summary, it is evident that the 4200 SM has been able to add multiple benefits to the mining process in regards to the numerous objectives at NAC compared to the CM system. The WSM system is associated with higher productivity, thus decreased machinery requirements leading to reduced emissions. Cost savings were identified in unit mining costs and CHPP operating costs. Coal loss and dilution are minimised, resulting in improved deposit recovery. Less machinery interaction on the work area indicates positive safety aspects. The WSM improves ergonomic prerequisites for the operator. Overall the 4200 SM implementation was very successful. All KPIs were fulfilled and often exceeded.

ACKNOWLEDGEMENTS

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TARGETED BUFFER BLASTING TO CONTROL MOVEMENT ALONG BEDDING PLANE SHEARS

John Latilla¹ and Batdelger Tumur-Ochir²

ABSTRACT: At the Ukhaa Khudag (UHG) coal mine, working part of the Tavan Tolgoi formation located in the southern Gobi desert of Mongolia, there have been minor to moderate slope failures in locations of relatively shallow overall slope angles. The majority of these events have been due to sliding along bedding plane shear zones that are generally associated with the coal seams. The bedding plane shears have low cohesion and friction angles.

For economic reasons not all seams are mined progressively down dip from the base of weathering, leaving some coal and overburden in situ up dip of the excavation. A solution is required to enable slopes to be mined at a steeper angle than the strata dip dictates. Targeted buffer blasting has been trialled with encouraging results.

Targeted buffer blasting is designed to disrupt identified plane(s) of weakness, disturbing them in order to increase cohesion and friction angle. The explosive charge weight per hole is generally significantly less than that used for a production hole of the same depth. Once exposed, the batter or slope will appear less damaged than it would in a normal buffer (or softwall) blast. A secondary advantage of buffer blasting is improved drainage, which lowers the phreatic surface.

Seven individual targeted buffer blasts have been analysed of which four have been classified as successful, two were probably successful and one was unsuccessful. The unsuccessful case was probably influenced by a nearby major blast.

INTRODUCTION

The Ukhaa Khudag (UHG) coal mine is situated approximately 250 km north of the border with China. The mine is a large, truck and excavator, open pit (terrace mining) operation producing medium volatile hard coking coal and high energy low sulphur thermal coal. In 2014 a total of 26.3 Mbcm of overburden was removed, allowing 4.6 Mt of ROM coal to be extracted. The operation is capable of significantly higher production rates, 15 Mtpa installed ROM capacity of the coal handling and preparation plant (CHPP) available, matched by available mining production fleet.

The mine is operated by Energy Resources LLC (ER), an indirect wholly owned subsidiary of the Hong Kong Stock Exchange listed Mongolian Mining Corporation (MMC). Thiess Mongolia LLC (Thiess) is engaged as the mining contractor. As at June 2015, the excavated pit measures approximately 2.2 km from the lowwall crest in the east to the highwall crest in the west, and approximately 2.0 km between the northern and southern endwall crests.

The pit is currently around 170 m deep and is planned to have a final depth of 350 m. The strata dips between 3° and 17° into the highwall (towards the west) while the flanks (endwalls) dip into the pit by between 5° and 40°. In order not to sterilise the lowest coal seam, which has yet to be mined, all overburden removed is currently dumped ex-pit.

There have been a number of sliding failures along bedding plane shears associated mostly with the coal seams. This has led to relatively flat overall slope angles (OSA) along the endwalls with a resultant reduction in productivity. A potential solution to this problem was identified as disturbing the bedding plane shears by blasting thereby increasing the friction angle and cohesion. This technique has been called targeted buffer blasting.

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GEOLOGICAL STRUCTURE AND STABILITY ISSUES

East – West structure

The pit is advancing in a westerly direction and the strata generally dips into the highwall by between 3° and 17° as illustrated in Figure 1, noting that three times vertical exaggeration has been applied to the cross section.

![Figure 1: Typical E-W cross section](image)

North – South structure

In the eastern and central portions of the planned pit, the strata forms a roughly flat bottomed basin structure in the N-S direction and the seams generally dip out of the pit walls on both the northern and southern endwalls (Figure 2). However, further to the west the coal bearing strata is cut off on the northern and southern flanks by faults. The seam dip is generally from south to north and is also much steeper, up to around 35°. The far western area is not considered in this investigation.

There has been a significant degree of folding and thrust faulting resulting in the bedding plane shears that occur. The structural history has been described in the Resource Estimation for Ukhaa Khudag Coal Mine, prepared internally by the ER Exploration and Geology department (MMC, 2012). The geological structure at UHG can be briefly summarised as follows:

Major compressional forces resulted in the entire UHG deposit being transported some distance along the basement contact before the movement stopped due to it encountering a buffer. The compressional forces would then have ramped up into the coal measures creating numerous thrust ramps and associated structures. This would include low angled thrusting within the weaker coal seams, observed as bedding plane shears in the coal.

Other disturbed zones (shears and faults) are present at UHG and can be up to tens of metres wide. These have a wide variety of dips and dip directions and tend to cut through bedding planes. They are most commonly associated with thrust faulting but some are probably due to strike-dip faulting. These features are not considered in this study.

Structural summary

The structure at UHG is complex with multiple disturbance phases. The dominant environment is compression, which results in faulting, folding and shearing. The northern and southern extents of the UHG deposit are generally bounded by major faults while there are numerous other faulted zones. The structure of these other faulted zones is such that although coal seams are present within them, they are hard to model or predict.
INFLUENCE OF BEDDING PLANE SHEARS

Bedding plane shears have been identified as being the major driver of significant sliding failures at UHG. The behaviour of the slopes is far more dependent on structure than on rock strength. About 90% of all significant failures have been classified as sliding along bedding shears.

Bedding plane shears at UHG are very often characterised by a weak brown clay fill or alternatively by finely pulverised coal. They can range in thickness from as little as 10 to 20 mm up to as much as 500 mm.

Figure 3 shows examples of some typical bedding plane shear zones at UHG:

A. steeply dipping shear (±40°) near base of coal seam.
B. shear plane after slippage has occurred, note polishing, this fill consists of powdery coal and disintegrates quickly on exposure to air.
C. ± 30 mm wide, clay filled shear in coal.
D. ± 150 mm wide soft clay filled shear at top of seam, some movement of the upper surface is suspected combined with spalling of coal. Obtaining good quality photographs of the pulverised coal filled shears is difficult as the fill has a sugary appearance and once exposed looks like very fine spalled coal.

While no direct testing has been done, zero cohesion and a friction angle of 13° have been assumed in models containing bedding plane shears at UHG. These values have been confirmed by back analysis and the friction angle is in line with those quoted in Barton (1973):

- Clays: over-consolidated, slips joints and minor shears with peak friction angle 12.0° to 18.5° and residual friction angle 10.5° to 16.0°.
- Coal measure rocks: clay mylonite seams 10 to 25 mm thick with peak friction angle 16.0° and residual friction angle 11.0° to 11.5°.

There have been recorded instances of nearby production blasts initiating failure as well as re-mobilising existing failures. This has been confirmed by crackmeter monitoring by the site geotechnical team. Apart from failures along bedding plane shears, the only other major failures recorded so far at UHG have been along unfavourably oriented fault planes, these are however not common.

POTENTIAL SOLUTIONS

At UHG the mining layouts are checked annually by an external consultant based on a set of approximately forty detailed cross sections provided by the site geotechnical team located around the pit. The principal cross section profiles analysed are the current pit walls and the planned pit wall profile.
at the end of the year. In addition, any unfavourable structures lying between the two profiles are analysed.

![Figure 3: Bedding plane shear examples](image)

It is at this stage that potential low factor of safety (FOS) slopes are identified. These may be the entire slope or, more commonly, a portion of the slope or an individual batter stack. Where the potential instability is due to the presence of bedding plane shears the following three remedial actions are usually considered:

- **Mine the coal from the top down following the seam dip where the dip is less than approximately 20.0° or terrace along strike where the dip is steeper.** The resultant slope angle is about the same as the seam dip or shallower. In many cases this is not optimal for coal recovery especially in tough financial times where it is necessary to target the most advantageous stripping ratio. In some instances this means that slopes containing unfavourably dipping bedding plane shears are left.

- **Potentially unstable slopes or batters may be controlled by forming a waste rock buttress at the toe.** However, UHG is currently constrained in that the lowest seam in the succession has yet to be mined, and as a result in-pit waste dumping is not able to be used on a routine basis.

- **Targeted buffer blasting is a third option.** This entails placing a relatively light charge in the vicinity of the zones containing bedding plane shears to “rough-up” the contacts. This has the effect of increasing the cohesion and friction angle of these zones and is the main focus of this paper. The effect of targeted buffer blasting is shown in Figure 4.

Buffer blasting disturbs the bedding plane shears, resulting in disrupted continuity along the shear zones. This will hold true in cases where the disruption is greater than the thickness of the shear zone. To be effective the blast must only be powerful enough to disturb the rock on either side of the shear and not pulverise the entire blast block.

It must be pointed out that the idea of blasting to disturb planes of weakness is not new:

- **The earliest example identified is in a civil engineering context.** A “shot-in-place rock buttress” was used in the late 1960’s to control a block glide landslide above a highway in Tennessee. Following this, a set of six further “shot-in-place rock buttresses” were formed between 1976 and 1980. These were all still stable in 1986 (Moore, 1986).
A more recent reference to a similar approach describes blasting of the overburden to turn a geologically disturbed section of overburden into a relatively homogenous block of blasted rock. The blasting removes the influence of jointing and faults and the overburden face is then battered back to about 45.0° with the dragline. This method is referred to as softwall blasting in Australia (Kelso, 2011) and is widely practiced in the Bowen Basin.

Figure 4: Low FOS slope stability improved by forming a buffer blast at the toe

BUFFER BLASTS FOR DIFFERENT PURPOSES

The type of buffer blast used at UHG depends on the condition that must be controlled, the two types used to date are bench buffer blasts and targeted buffer blasts. Buffer blasting at UHG is understood to be any blast where the blasted material will largely be left in place to form the batter or slope face.

- A targeted buffer blast strip (shot-in-place buttress) is utilised where a target zone, usually a coal seam containing bedding plane shears, has been identified. The intention of the targeted buffer blast is to disrupt the bedding plane shears at seam level and then displace the rest of the overlying strata without completely fragmenting it. This technique is generally applied where the seam dip lies between 5.0° and 20.0°.
- A bench buffer blast (softwall) is used where the entire batter face and the bench behind it is assessed as being so structurally disturbed that it is better to blast it and obliterate all structure, as far as is practical. The batter and bench are blasted with a similar charge weight as a normal production blast and the blasted material is battered back to between 40.0° and 45.0°. This method would generally be used where the majority of structures are dipping at over 20.0° and is not covered in this paper.

MATERIAL PROPERTIES OF BLASTED ROCK IN SITU

No direct tests have been carried out at UHG to determine the shear strength properties of blasted rock left in situ. Rock properties and strengths used for limit equilibrium analyses at UHG have evolved with time and the values currently used are as follows: Unit weight 21.4 kN/m³, cohesion (c) = 60 kPa and friction angle (φ) = 33.0°. These values are based on the following from the literature:

- Bowen Basin unsaturated cat 4 Spoil (Simmons and McManus, 2004) where c = 50 kPa and φ = 35.0°
- Bowen Basin softwall (Kelso, 2011) φ = 30.0°
- Bowen Basin softwall (communications with site geotechnical engineers) c = 50 to 100 kPa and φ = 35.0°
- Tennessee, in jointed sandstone with shale bands, (Moore, 1986) φ = 38.0° and unit weight = 22 kN/m³.
PHREATIC SURFACE

The locality of the phreatic surface (water table) is critical in producing a valid limit equilibrium analysis. At UHG the phreatic surface model has been derived by dipping the water level in selected blast holes prior to charging up. The following simplified model for the average water depth below various pit wall features is as follows:

- At surface = 23 m
- Below bench or batter crests = 15 m
- Below bench or batter toes = 6 m
- Below overall slope toe and under pit floor = 1 m

Where a buffer blast is present the assumption has been made that the buffer blast zone is freely draining. The phreatic surface is therefore assumed to conform to the base of the buffer blast.

LIMIT EQUILIBRIUM ANALYSES

Galena (Clover Technology 2015) has been used for limit equilibrium analyses at UHG. Models are built by tracing geological cross sections supplied by the site geotechnical team and in some cases these cross sections are very complex. Occasionally the models have used up all fifty material profiles allowed in Galena. Examples of Galena models are shown in Figure 4.

Buffer blasts are limited to a maximum depth of 40 m in the design stage due to equipment limitations. However, occasional buffer blasts to a depth of around 50 m have been conducted. The aim of the blast drilling is to intersect known, or suspected, zones of bedding plane shears.

The required width of the buffer blast is arrived at iteratively by modelling in Galena and targeting a minimum FOS of 1.2. In some cases it is necessary to add a supplementary waste rock buttress to achieve the target FOS. The length of the buffer blast is derived by considering a set of cross sections along a section of the slope (Galena is a two dimensional code).

Limit equilibrium analyses at UHG have indicated that:

- Targeted buffer blasting to control movement along bedding plane shears is a practical option within the seam dip range of 5.0° to 20.0°.
- No sliding along bedding plane shears is expected where the seam dip is less than 5.0° and no purpose would be served by targeted buffer blasting in this range.
- Practical limitations indicate that targeted buffer blasting will be difficult where the dip exceeds 20.0°. At steeper dips the option of extracting coal along dip from the top down should be considered. Alternatively, the entire affected slope may be bench buffer blasted in a series of 50 m high batters.

BUFFER BLAST LAYOUT AND DESIGN

Issues to be considered when designing a targeted buffer blast include:

- Drilling equipment capabilities (maximum practical drill hole depth) which determines whether the targeted bedding plane shear zones can be intersected
- The berm or bench where the holes must be drilled must be suitable for safe drilling, i.e. wide and flat enough.
- Whether angled buffer blast holes will give better access to suspected zones of slippage (not yet done at UHG but likely to be as successful as vertical holes).
- Scheduling: occasionally it is too late to buffer blast an area because the overburden has already been blasted. For a targeted buffer blast the charge weight is typically around 40% of the normal charge weight used for production blasts of the same depth in the same area. Some typical blast design parameters for UHG are as follows:
Burden = 7.5 m to 8.0 m (typical)
Spacing = 9.0 m to 9.5 m (typical)
Drill hole diameter = 229 mm (typically)
Average charge weight per blast hole:
  - Deep holes (≥ 40 m) = 1,010 kg
  - Intermediate holes (20 to 40 m) = 332 kg
  - Shallow holes (<20 m) = 316 kg
Average powder factor = 0.36 kg/bcm (with range 0.14 to 0.52)
Maximum instantaneous charge (MIC) initiated within 8 ms duration = 2,532 kg.

RESULTS
A total of seven UHG buffer blasts have been assessed for effectiveness in this study, four were successful, and two were classed as probably successful while one was unsuccessful. The results are shown in Table 1.

Table 1: Summary of successful and unsuccessful cases

<table>
<thead>
<tr>
<th>Blast Block</th>
<th>Pit Sector</th>
<th>Date</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>586a</td>
<td>NEW1</td>
<td>3/09/2013</td>
<td>Probably unnecessary in retrospect, as flat seam dip was identified in a subsequent (closer) cross section. Indicated dip at time of design 6.0°.</td>
</tr>
<tr>
<td>605</td>
<td>NEW1</td>
<td>25/09/2013</td>
<td>Successful (without subsequent placement of waste buttress).</td>
</tr>
<tr>
<td>397</td>
<td>ELW</td>
<td>3/12/2012</td>
<td>Successful (ramp operating on top of buffer block - no cracks observed).</td>
</tr>
<tr>
<td>433</td>
<td>NEW1</td>
<td>4/02/2013</td>
<td>Successful.</td>
</tr>
<tr>
<td>480</td>
<td>NEW1</td>
<td>23/03/2013</td>
<td>Successful.</td>
</tr>
<tr>
<td>512</td>
<td>SEW1A</td>
<td>6/05/2013</td>
<td>Unsuccessful (major endwall failure, triggered by box cut blast, overran buffer strip). Waste buttress not placed on top. Buffer may have prevented the failure from extending further down slope.</td>
</tr>
<tr>
<td>675</td>
<td>SEW1A</td>
<td>26/11/2013</td>
<td>Probably successful - slope behind buffer stable but narrow strip between buffer and toe is unstable (where they overlap) - floor heave at toe.</td>
</tr>
<tr>
<td>343</td>
<td>SEW1</td>
<td>12/10/2012</td>
<td>Probably successful (slope stable but exposed buffer portion of slope does not appear very disrupted)</td>
</tr>
</tbody>
</table>

Site personnel report that there were no cases where:

- A buffer blast was recommended but not implemented and then the slope failed
- Recommended buffer blast was not done but the slope remained. Mining personnel, when asked, were of the opinion that buffer blasting helps the slope stability because of the result of the successful buffered slopes, especially those along the Northern Endwall.

The case classified as unsuccessful (Blast Block 512) requires additional discussion. It is possible that this buffer blast did assist stability to some extent but there is no doubt that the major south endwall failure which occurred upslope from this buffer blast strip overran the buffered area. Crackmeter monitoring indicated that slope movement was triggered by a nearby, high energy, confined production blast. In addition to the high energy blast nearby, a 6 m high by 10 m wide waste buttress planned for placement on top of the buffer strip was not constructed.
One indication that the buffer blast was at least partially effective was the absence of signs of floor heave on the downslope side of the buffer blast strip indicating that bedding plane shear movement was probably arrested by the buffer blast.

The geometries and FOS values for the buffer blasted slope areas are summarised in Table 2.

Table 2: Buffer blast slope FOS and geometries

<table>
<thead>
<tr>
<th>Blast Block</th>
<th>Pit Sector</th>
<th>Seam Dip (ψ₀) (°)</th>
<th>Slope angle above buffer blast (ψ₁) (°)</th>
<th>Angle between slope face and seam dip* (ψ₁-ψ₀)</th>
<th>FOS</th>
<th>Design dimensions</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>586a</td>
<td>NEW1</td>
<td>2 to 6</td>
<td>15</td>
<td></td>
<td>1.25</td>
<td>40</td>
<td>10</td>
</tr>
<tr>
<td>605</td>
<td>NEW1</td>
<td>8</td>
<td>20</td>
<td>12</td>
<td>0.88</td>
<td>27</td>
<td>40</td>
</tr>
<tr>
<td>397</td>
<td>ELW</td>
<td>15</td>
<td>NA</td>
<td></td>
<td>0.64</td>
<td>22</td>
<td>38</td>
</tr>
<tr>
<td>433</td>
<td>NEW1</td>
<td>5</td>
<td>27</td>
<td>22</td>
<td>1.17</td>
<td>50</td>
<td>10</td>
</tr>
<tr>
<td>480</td>
<td>NEW1</td>
<td>5</td>
<td>16</td>
<td>11</td>
<td>1.01</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>512</td>
<td>SEW1A</td>
<td>5 to 11</td>
<td>18</td>
<td></td>
<td>1.14</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>675</td>
<td>SEW1A</td>
<td>9 to 12</td>
<td>13</td>
<td>2.5</td>
<td>0.62</td>
<td>32</td>
<td>30</td>
</tr>
<tr>
<td>343</td>
<td>SEW1</td>
<td>5 to 10</td>
<td>24</td>
<td>16.5</td>
<td>0.81</td>
<td>22</td>
<td>43</td>
</tr>
</tbody>
</table>

* Only successful and probably successful cases, seam dip average values used
** Ramp constructed over buffer blast remains stable. Slope above ramp 28° but in a different geotechnical domain.

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SLOPE FAILURE INFLUENCED BY BLASTING

The extensive failure on the southern endwall which overran the buffer blast strip (unsuccessful case) is suspected to have been influenced by nearby blasting. A confined, high energy production blast in the box cut sited on the western end of the failed area was shown by crackmeter monitoring to have led to acceleration of sliding which culminated in the slope failure. The blast in question, Blast Block 547, had a MIC of 4,604kg.

Considerable work was subsequently done on site to quantify slope damage due to blast vibrations but will not be covered in detail in this paper. Intact rock was expected to be damaged for a distance of up to 150 m from the blast edge and a single blast of the magnitude of Blast Block 547 was estimated to be enough to cause failure up to about 50 m away, as discussed briefly below.

Naismith (1984) indicates that peak particle velocity (PPV) values for a confined (box cut) blast may be as much as three times that of a blast with at least one open face. The same paper quotes the following damage criteria from other authors:

- Oriard (1972) states that falls of loose rock can occur between 50 mm/s and 100 mm/s, partially loosened sections (both underground and on surface slopes) can occur from 130 mm/sec to 380 mm/sec, while damage to intact rock is expected over 635 mm/sec.
- Kiel and Burgess (1977) conclude that the formation of new cracks occurs from 305 mm/sec to 610 mm/sec.

In the absence of seismograph data for blasts at UHG, the PPV was estimated using the following equation, suggested in Müller et al (2007) to determine the PPV for sedimentary rocks:

$$\text{PPV} = k \times L^0.6 \times r^m$$
Where:

\[ L_B = \text{Charge weight per delay or MIC (kg)} \]
\[ r = \text{Distance between blasting point and point of interest (m)} \]
\[ k = \text{constant of 969, modified to 1,410 subsequent to observation of blast damage} \]
\[ m = \text{constant of -1.51} \]

Using the equation above including update where \( k = 1410 \), the PPV at a distance of 50 m from the blast block edge is estimated as 600 mm/sec dropping to 115 mm/sec at a distance of 150 m. As this was a confined blast, the PPV values may have been as high as 1,800 mm/sec at a distance of 50 m and 345 mm/sec 150 m away.

UHG now designs endwall blasts and those close to endwalls, so that at critical structures (e.g. bedding plane shears dipping >5°) the forecast PPV is not to exceed 130 mm/sec within 100 m of the Blast Block.

**CONCLUSIONS**

It should be noted that this is a relatively small sample of cases and as such the following conclusions should be treated with caution:

- In 86% of cases studied, the buffer blasts have been successful or probably successful in stabilising the slope.
- Blast vibration has influenced movement in some cases. This has received significant attention on site and is far better controlled now.
- It appears that, on average, a slope of up to 13.0° above the strata dip (\( \psi_f - \psi_p \)) can be maintained with the aid of buffer blasting. There is as yet, insufficient data to warrant any statistical analysis of the results.
- Conditions at UHG are generally quite dry, rainfall is low and there are no strong aquifers so the effect of water may lead to different outcomes elsewhere.

**ACKNOWLEDGEMENT**

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**REFERENCES**


EFFECT OF PUNCH LONGWALL RETREAT ON HIGHWALL STABILITY

Luke Clarkson

Abstract: Punch longwall mining takes advantage of the final highwall exposure in an open cut coal mine, driving gateroads straight into the targeted coal seam then retreating the longwall back to just short of the highwall. A designed barrier pillar remains between the final longwall position and the highwall, which is subject to redistributed ground stresses. Material is strained to magnitudes unlike those typically measured in open-cut mining, and unlike typical longwall ground behaviour, the highwall was observed to strain in an opposite direction to the longwall caved and subsidence zone. Mine personnel and equipment may become exposed to the unstable highwall rockfall hazards, highlighting the importance of understanding the mechanism and implementing appropriate controls.

This paper describes the assessments undertaken through radar, survey prism and Light Detecting and Ranging (LiDAR) monitoring, as well as geotechnical inspection and analysis. Initial results show highwall movement is directly correlated with longwall goafing and any delayed ground movements are more related to the rate of retreat, the type of goafing behaviour and the influence of strata deteriorating in other locations along the face of the highwall. This paper also describes climatic conditions as the primary limitation with radar monitoring. In addition, diurnal steel mesh movement was measured with changing atmospheric temperature. Steel mesh is usually installed as ground support above portal entries to the underground workings. By filtering out any measured mesh movement, true trends of strata deformation in these areas can be identified. Recommendations are made for efficient and reliable radar data acquisition. Further recommendations are made to restrict people and plant access to a safe standoff from the highwall as the longwall approaches the final retreat position within the panel. Effective monitoring of highwall performance throughout the longwall retreat to establish stable trends will enable continued, safe access to these locations.

INTRODUCTION

Broadmeadow Coal Mine located in Central Queensland, Australia, operates the punch longwall underground mining method. Roadways are driven into the coal seam from previously excavated open cut operations. The open face of the final highwall can creep and strain in line with open cut geotechnical practice and analysis. In the punch longwall method, the complexity of geomechanical behaviour is heightened with longwall excavations undercutting the ground, at depth behind the highwall face. Typical overburden caving above the longwall usually reduces the competency of the major geological units, inducing ground stress redistribution and surcharging on the residual pillar immediately behind the highwall, as identified in Figure 1.

Although highwall stability is retained, the geomechanical behaviour related to longwall retreat can induce ground deformations much greater than those generally accepted in open cut stability assessments (Sullivan, 1993). As such, it is important that the mine operator understands the significance of these events to adequately manage the safety of personnel and equipment while ensuring productivity requirements.

Ground monitoring practice at Broadmeadow has now provided the opportunity to investigate trends, correlations and relationships of highwall and deep-seated deformation to mining practice and external factors. This paper presents a back-analysis of radar and survey monitoring data in order to gain a greater understanding of the behaviour at hand.
MINING AND DATA CAPTURE

The barrier pillar width at Broadmeadow ranges from 95m to 225m, with an average of 142m. The mine plan and cross-sections of the region addressed in this paper is observed in Figure 1.

An SSR-XT radar was used for monitoring as the longwall neared the final stages of retreat. Data capture commenced when the longwall was, on average, 161m from the open face. This instrument has the capability to scan a distance of 30m to 3500m away from the radar setup, identifying failures to a resolution of 0.3m x 0.3m and 30.5m x 30.5m, respectively. At the reporting distance of 215m presented in this case, the integrated visual imaging system resolves a 2m x 2m pixel (Figure 2).

Historically, geotechnical engineers and Explosion Risk Zone (ERZ) controllers routinely monitor to understand and manage the risk to personnel and equipment from isolated rock falls, failed rock bolts,
and inadequate drainage control on catch benches. These visual controls are established and implemented regardless of observed movement.

With any visual inspection, the requirement to capture early movement is based on the experience of the engineer or Statutory Official. BMA has supplemented these visual inspections with three different primary forms of survey monitoring, to differing resolutions. Each may be successfully used for monitoring if these resolution errors are accounted for. Firstly, prisms are embedded on the highwall for east-west movement. Secondly, stakes over the crest of the highwall identify vertical movement. Thirdly, airborne Light Detecting and Ranging (LiDAR) monitoring quantifies subsidence over the panel through laser survey of surface topography. Considerations to be made which may affect monitoring resolution through each method are seen:

Prism:
- Typical resolution of ± 3mm;
- Provides best measurement of movement out from wall;
- Relies on survey instrumentation;
- Limited sample compared to scan/ radar however accuracy is at the upper end of radar’s capability; and
- Limitation on time-based coverage and efficiency of technology in monitoring.

Stakes:
- Typical resolution of ± 10mm; and
- Relies on human eye and associated limitations.

Airborne Light Detecting and Ranging (LiDAR):
- Typical vertical resolution of ± 0.2mm;
- GPS to control position of airborne plane mounted with laser scanner;
- Inertial Navigation System (INS) unit in plane to correct tilt and roll;
- Methods can be employed to increase resolution to maximum ± 0.1mm; and

Acknowledging system capacity and existing limitations to allow a greater understanding of the strata behaviour is critical in stability assessments. Effective monitoring of highwall performance throughout the longwall retreat to establish stable trends will enable continued, safe access to these locations.

THEORISED MECHANISM

To understand the stress redistribution with the undercutting of the highwall block, a generalised model was developed in Phase2. The model acts under the following conditions (Figure 3):

- Principal horizontal stress = 1.7 x vertical stress in plane and 1.3 x vertical stress out of plane everywhere aside from coal seam (1.0x).
- Assumption of continuous, homogeneous, plastic materials. Plastic conditions are inclusive of residual parameters.

![Figure 3: Model of Theorised Mechanism](image-url)
Comparisons of stress distribution for different extents of staged caving are shown in Figure 4 and Figure 5.

**Figure 4:** Finite Element Analysis on Sigma YY (Normal Stress in Y direction, Vertical Stress)

**Figure 5:** Finite Element Analysis on Sigma XX (Normal Stress in X direction, Horizontal Stress)

Phase 2 finite element modeling indicates maximum stress changes at the immediate highwall of -0.94 MPa vertically, and -0.47 MPa horizontally (compression positive notation). The inference here of a beam-like caving mechanism to the east of the longwall face is supported, where Sigma XX reflects stress trending with a maximum compression at the top of the beam, and tensional at the base, indicating sag of the sandstone unit. Once this mechanism is understood, one can interpret Sigma YY to exhibit the same behavior, where tensile stresses nearer the excavation indicate caving into this area, and overburden stresses concentrated at the center of the sagging units generate increased stress in the upper units. Sigma XX and Sigma YY demonstrate purely horizontal and vertical stresses, respectively, without the influence of stress orientation change post-excitation.
Lipping, identified as the differential lateral displacement between adjacent stratigraphic layers, was observed on the highwall and is thought to result from shearing between the layers. This is mainly supported by the smoothness of the exposed surfaces, suggesting shearing of the pre-existing asperities. This can occur when the shear stress between the layers exceeds the binding shear strength (Ritter, 2002). The magnitude of lipping is dependent on the extent of shear stress exceedance and the distribution across the layers (Figure 6).

![Figure 6: Magnitude of Lipping for Different Stratigraphic Interfaces and Photo of Lipping between Stratigraphic Layers (P Seam)](image)

As lipping occurs, greater strain is induced on the strata above the layer boundary, which induces the tendency for blocky failure and strain to be carried back into the strata.

Visual monitoring indicates an inversely proportional trend between strain magnitude induced on the highwall and distance between the longwall face and final longwall position \( \varepsilon = \frac{1}{d} \). Strain on the highwall face is measured as proportional to the height of the target area from the ground.

Longwall retreat through the coal seam (Figure 2) undercuts the overburden strata, altering the stress field and orientation as well as inducing a displacement of the overlying units relative to the excavation. One visible form of this is surface subsidence above the longwall panel. The remainder occurs within the strata units, which cave behind the longwall face and into the goaf (longwall goafing). Hydraulic supports (longwall shields) are utilised across the span of the longwall face in order to support the strata above the current shearer position, protecting both equipment and personnel as mining progresses and the longwall retreats toward the open face.

Retreating at a faster rate has two primary influences; both related to the time-dependent deterioration of overburden strata. Firstly, it reduces the detrimental impact of load build-up on shields, their serviceability and any potential of becoming iron-bound ('buried' after excessive convergence). The second influence is whereby behaviour of the overburden strata is emphasised, and self-spanning competency of units can cause hang-up of the strata, leading to larger magnitude falls as opposed to periodic caving behaviour. The influence on the longwall supports in this case is minimized where the supports, with adequate retreat rate, progress past the fracture point ahead of the face before extensive time-dependent strata influence will be transferred. It is because of these two factors that speed of longwall progression is not only favourable for production, but also in limiting the detrimental influence of strata on longwall shields. This is stated with no other external considerations on retreat rate present, such as hydraulic fracturing at optimal times. The practicality of this concept is discussed in *Deformation vs Shears*.

LiDAR analysis, with output observed in Figure 7, showed the last 100m of strata, measured east from the open face exhibited no vertical displacement. This indicates that known displacement of the strata was purely horizontal. Voids in this plot relate predominantly to surface works which distort the real data.

Modelled total displacement vectors in Figure 8 show a distinct trend of strata movement fanning out from the subsidence and caving zone into horizontal movement, directed toward the open face of the highwall. The direction of deformation is greater inclined to the open face proportionally with distance to the west. The blue curve indicates the first trend of strata deformation which is directed beyond the excavated void and into the barrier pillar, commencing 67m east of the highwall toe on the surface.
Modelled vertical and horizontal displacements were also generated in Phase 2 (Figure 9). The intention of reporting the deformation model is to determine the behaviour, rather than absolute values. In introducing residual values through the assumption of plastic behaviour, the finite element model for this scenario has heavily over exaggerated the magnitude of deformation in the caving zone compared to that known, and realistic in the actual environment. Cooler colours represent lower values, hotter represent greater. Zero values are red for vertical displacement and green for horizontal displacement, and are consistent across the highwall prior to excavating underneath. The contour plot highlights that predominantly negative values are applied to the deforming strata for vertical displacement.

The horizontal displacement plot shows a tendency for the strata to move outward at the location of the exposed face. The theoretical angle of draw, taken from the vertical displacement plot ahead of the longwall face equates to 13.5°, significantly less than site approximations of 28°. An exploded view of the horizontal deformation theorised to occur on the immediate highwall face is shown in Figure 10.
Figure 10: Exploded Horizontal Deformation on the Immediate Highwall Face

Figure 10 shows a trend of heightened deformation of the massive sandstone unit out from the wall, and movement into the wall nearer the crest of the highwall. Any horizontal movement along the stratigraphic boundary is anticipated to reflect lipping behaviour. The behaviour observed only approximates that modelled, as seen when comparing this data to Figure 6, where the boundary between coal and massive sandstone reflects the coal layer lipping beyond the sandstone. The data in Figure 10 is complemented by analysis of the deformation vectors, seen in Figure 8, where there is a defined ‘zero-line’ whereby the influence of the goaf no longer draws on the strata to the west.

DEFORMATION vs SHEARS

There is measured correlation between highwall movement out from the wall and the rate of retreat on the longwall. The data in this section is based on averaged readings over the mid-longwall face region highlighted in Figure 11.

Figure 11: Highwall Scan Area to be Analysed (Green Shade)

An investigation into the rate of retreat against the rate of highwall movement is discussed in this section. Two primary regions of trending data are defined. Section (a) of Figure 12 is a region of higher retreat rate and coincident with higher highwall deformation rates. Section (b) is a region of a steady rate of retreat and a near-linear deformation trend.

Figure 12: Chart Showing Shearer Position vs Deformation over Time
An average was taken over Section (b) in order to distinguish the total movement per longwall shear, and over a longer average time. The average over a 24 hour segment was taken. Interpreted rates are calculated as 2.28mm of movement per shear, and 3.36mm of movement per day. Direct comparison between these two rates indicates that movement predominantly occurs as a result of retreat, which emphasises the influence of the underground longwall excavation with movement of the highwall.

Section (a) must also be analysed to determine the relationship between increased shearer speed and the associated deformation. A focus on deformations and velocities throughout this section is provided in Figure 13.

The sandstone unit, when unbroken, is of greater length and hence volume as strata competency (self-spanning capacity) combines with heightened rates of retreat to extend the void underneath. Eventual fracturing induces stress redistribution to overburden layers and consequent gradual deterioration of these units, with the effects observed primarily through subsidence and stress redistribution of adjacent units is in line with Newton’s third law. This behaviour is not instantaneous, but rather observed through delayed deformation, where magnitude of deformation and delay time are dependent on strata characteristics. It is suggested that the fracturing of the self-spanning sandstone units is the primary cause for increased movement rates (Figure 13). An analysis of a greater number of events would further validate this theory.

A direct analysis of goaf behaviour can be taken by assessing Longwall Visual Analysis data, which records and documents a number of loading parameters through sensors on the hydraulic roof supports (longwall shields) positioned along the face (Figure 14).
Based on site experience, the periodic increase in time weighted average pressure is a function of the geometry and spanning capacity of the overburden cantilevering into the goaf. Once this cantilever fractures and joins the goaf, the shields reflect a decrease in pressure which builds again as the next cantilevering unit is supported. In some cases this can cause more significant longwall weighting events as was experienced between 28th July and 29th July.

A direct correlation between longwall caving behaviour and highwall deformation is suggested. Measured increased deformation on the tertiary layers above the immediate highwall face supports this theory, where deformation occurs in line, albeit delayed deformation, with shields being removed from the final longwall position. This occurs from north to south, parallel to the highwall face (Figure 15).

Figure 15: Rate of deformation vs time for tertiary layer as shields are removed

Figure 15 shows that any increase in rate of movement occurs at the same time for each set of shields, no matter the position along the face. It is the magnitude of the rate of movement during these increased periods which is worthy of noting. It is assumed that each increase in rate of movement is reflective of a goaf convergence event where the removed shields once stood. The amount of rock falling into the goaf is anticipated to be proportional to the magnitude of rate of movement in Figure 15. Dependent on proximity to the fall area, either primary abutment loading or secondary effects on deformation are measured on the highwall face an average of two and a half days after the associated goaf has fallen. This aligns with the proposed relationship between cave behaviour and highwall deformation as discussed in Figure 13.

It is suggested that the abutment support of the pillar and unmined longwall panel to the south causes shields nearer the tailgate end of the face to experience both a lower rate, and a lower total magnitude of deformation when compared against the response of other shield locations across the face.

In consideration of the variability of strata geomechanics and characteristics, the recommended operational control is to restrict people and equipment access to outside a safe standoff zone while the longwall is cutting, with close monitoring during, and until trends stabilise afterward. The best estimate on controlling accelerated highwall deterioration at this stage is applied where movement trigger rates of 1.5 mm/2 hours and 2.0 mm/2 hours over two radar scans are implemented above the portal and sump, respectively. The two hour period allows greater responsiveness to operational changes, yet includes enough data points to eliminate noise (such as diurnal, and atmospherics). It is important to note that alarm settings used should be based on trigger points that would suggest the slope movement has changed from a steady state to a state of accelerated movement, potentially heading to failure. The triggered alarm settings that are quoted are the upper limit of what is currently accepted as steady state movement of the slope at Broadmeadow. The forward influence of goafing on abutment load, stress,
and strain promotes the recommendation for radar monitoring to be implemented when the longwall is within 200m of the zero line.

**DIURNAL MOVEMENT**

Diurnal movement relates to the deformation in radar monitoring from thermal expansion and contraction of steel mesh over the highwall. At Broadmeadow, heavily galvanised mild carbon steel mesh is installed above the portal entrances to control and redirect any fall of surface rock into catch drains.

As common with most material properties, mild carbon steel expands with increasing temperature. The theoretical magnitude of linear change is quantified through the coefficient of thermal expansion.

Characteristic properties of mild carbon steel give the coefficient of linear thermal expansion in the range of $11$ to $13 \times 10^{-6} \text{[m/(m * K)]}$ at 20°C. Working under the given assumptions, a relationship between change in temperature and the mesh behaviour can be defined (Thermal Expansion 2002).

**Assumption:**
- Coefficient of expansion does not change with temperature increase

\[
\frac{\Delta L}{L} = \alpha_L \Delta T
\]  
(1)

\[
\frac{\Delta L}{L} = 12 \times 10^{-6} \Delta T
\]  
(2)

where:

\[\alpha_L = \text{linear coefficient of expansion as a function of temperature } T\]

\[\frac{\Delta L}{L} = \text{fractional change in length}\]

\[\Delta T = \text{change in temperature (°C or K)}\]

In solving for actual strata against mesh deformation, consideration is made for the change in length of the mesh and effect of the length of mesh provided per metre on the highwall. This calculation would require the assumption that the mesh experiences this deformation averaged over the scanned metre, as displayed in Figure 16. Measured behaviour shows that with an increase in climatic temperature, the mesh will sag into the wall. On cooling, it retracts.

![Figure 16: Mesh deformation, (a) undeformed mesh (b) real deformation (c) assumed deformation](image)

Using the linear coefficient of expansion of $12 \times 10^{-6} \text{[m/(m * K)]}$ the expression is given:
\[
\frac{\Delta L}{L} = \alpha \Delta T \\
L = 12 \times 10^{-6} \times \Delta T \quad (3)
\]

As this extension is averaged over a metre length of mesh, it is assumed that \(\Delta L\) is also the equivalent of the lateral movement of the mesh.

By plotting the two variables against the actual deformation charts, the difference between actual and theoretical deformation can be observed. Actual deformation takes an average of the masked area between two vertical bolts. This is seen in Figure 17.

![Figure 17: Chart Showing Change of Temperature on Heating and Cooling Extremes against Change in Deformation in Individual Scans](image)

The sign convention of theoretical deformation has been reversed in order to represent thermal expansion of the mesh as a negative reading, aligning with what is known as real and described in Figure 16. The theoretical data is derived from the same periods of heating and cooling as defined for unstable conditions. It is observed that the rate of deformation in unstable conditions is approximately 3.7 times the theoretical rate for heating, and 17 times the rate for cooling. Limitations of the theoretical method in calculating mesh movement are identified where, in theory, this movement would be equal to the real, stable data. Theory also states that any strata instability would be identified through the difference between the rate and magnitude of deformation in the real values. The scatter of data and consequent coefficient of determination \(R^2\) values in the real, stable data deem it unreliable to compare against. Further, logarithmic and polynomial data trend lines presented a correlation more aligned than linear, yet still inaccurate in representing diurnal mesh influence. As such, the best estimate for strata deformation with diurnal variation is through isolating temperature cycles, with consideration of theoretical mesh deformation as plotted in Figure 18.

![Figure 18: Chart showing difference in actual vs theoretical mesh deformation over time](image)

It is the difference between the actual cooling and heating that defines the strata behaviour, as physical highwall deformation is applied regardless of the temperature cycle. This difference is dependent on the
change in temperature and its influence on diurnal mesh movement, an independent variable. If actual cooling is greater than actual heating, there is an outward movement of the wall and vice versa. The magnitude of difference gives an indication of strata movement (Figure 18). Within the stable zone, near-equal deformation is expected and the remainder of deformation spikes represent real movement.

WET WEATHER

A coherence event occurs on comparison between the change in range and amplitude of signal waves between one scan and the next. Two spikes are identified in the data presented in Figure 19. These spikes are a reflection of the external, unpredicted presence of rain on the highwall face, which distorts the data for a single scan.

Figure 19: Coherence event and influence on deformation trend with spikes on 30 August and 12 September

Once identified, the trending data can still be monitored in order to assess deteriorating conditions, albeit at a rougher accuracy while the presence of rain remains. If onset of instability were to be induced during the initial stages of a rainfall event, it is anticipated that the trend prior to the spike in data would be upward and alarming. As such, geotechnical monitoring of the highwall face can still be deemed effective with analysis of the trends either side of the spike in data.

CONCLUSION

Punch longwall retreat redistributes ground stresses toward the open face as a result of undercutting the material body. The highwall was observed to strain away from the longwall caved and subsidence zone, inducing a displacement of material outward up to 200m on the immediate highwall face. The following controls are recommended to protect personnel, plant, and operational safety:

- Radar monitoring when longwall is within 200m of the final position;
- Respective movement trigger rates of 1.5mm/2 hours and 2.0mm/2 hours over two radar scans above the portal and sump, dependent on designated steady state movement limits; and
- Recognition and consideration of monitoring limitations including wet weather and radar’s initial correction source.

Consideration of structure, seam and stratigraphic dip, as well as barrier pillar and highwall design is recommended when referring to different longwall panel retreats. The influence these parameters have on both caving behaviour and strata response is highly variable and generally unknown based on the number of case studies available.
Through knowledge and understanding of the interaction mechanism between underground and open cut workings, considerations can be applied in mine design to ensure safe access to underground workings, without interruption to operational activities.

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A REVIEW OF HIGHWALL MINING EXPERIENCE AND PRACTICE

Sungsoon Mo, Chengguo Zhang, Ismet Canbulat and Paul Hagan

Abstract: The highwall mining method originated in the USA in the 1940s. The aim of the method was to recover coal from a surface mine that has reached its economic limit. The coal is accessed at the base of the highwall from where a series of parallel entries are driven into the coal seam. Since 1991, highwall mining has been commercially used in Australia in approximately 40 pits at 17 coal mines (Christensen, 2004). Highwall mining provides opportunities to extract additional reserves with high productivity compared to underground operations and conventional surface operations. Therefore, some of open cut mines may consider adopting highwall mining as an alternative mining system when uneconomic conditions are expected due to higher stripping ratios. This paper reviews the highwall mining experience and design practices with an emphasis on geotechnical considerations. In addition, the relevant design issues for future research topics and challenges on highwall mining are discussed to enhance both the productivity and mine safety from the geotechnical point of view.

INTRODUCTION

Highwall mining refers to a mining method to recover coal from a surface mine that has reached its economic limit. The coal is accessed at the base of the highwall from where a series of parallel entries are driven into the coal seam. The entries are mined using two types of highwall mining systems, namely, continuous highwall mining (CHM) and auger mining (Figure 1). CHM system utilises a continuous miner generating typically 3.5 m wide rectangular entries while auger system excavates single or double holes typically from 1.5 to 1.8 m in diameter (Duncan Fama et al, 2001). The penetration depths range from 50 to 500 m (Verma et al, 2013) depending on the highwall mining systems and mining conditions. The pillars are then left between the mined entries to support the overburden.

Figure 1: Highwall mining systems (a) CHM system (after Caterpillar Inc.), (b) Auger mining system (Coal Augering Services 2014)

Highwall mining has several advantages compared to underground operations and conventional surface operations. Firstly, due to its flexibility and mobility, it is easier to recover smaller blocks of coal, which allows additional coal recovery from a final highwall or in constrained areas such as service corridors, spoil heaps and rivers (Shen 2014; Kleiterp 2010 and Fan, 2015). Secondly, highwall mining method is cost competitive as only three to seven crew members are needed (Luo 2013 and Schmidt 2015) and no roof bolting is required for the entries. Capital cost is less than underground mining with the average production capacity of in excess of 1 Mtpa (Shen 2014). Moreover, highwall mining is a safe mining...
method. Because the operators remain out of the entries they are not exposed to hazardous events such as roof and rib falls, mine fires and coal dust within the entries (Luo 2013).

Currently, the auger mining method can penetrate up to 200 m long and produce up to 60,000 tonnes of coal per month while the CHM method can drive up to 500 m and produce up to 124,000 tonnes per month (Shen 2014). Due to its longer penetration depth and productivity, CHM systems are preferred and more widely used than auger mining (Shen 2014 and Luo 2013). Even though a single CHM system rarely exceeded a production rate of 1.25 Mtpa in practical, its highest capacity of 1.5 Mtpa was proposed with upside of 2.0 Mtpa (Christensen 2004).

Therefore, some of open cut mines may consider adopting highwall mining as an alternative mining system when uneconomic conditions are expected due to higher stripping ratios or equipment availability. This paper reviews the highwall mining experience and design practices with an emphasis on geotechnical considerations. In addition, the relevant design issues for future research topics and challenges on highwall mining are discussed to enhance both the productivity and mine safety from the geotechnical point of view.

**HIGHWALL MINING EXPERIENCE**

**History of highwall mining**


In the 1940s, highwall mining method originated in the USA to mine additional coal from outcrops in contour mining, as a type of auger system. The first auger system excavated a single hole of 1.5 m in diameter and 30 m long, and recovered 726 tonnes of coal per day with a four-man crew. While the auger mining system was regarded as a new productive mining method, some problems encountered with the increased penetration depth of up to 120 m. The operations had to cease when a coal seam became thinner than the diameter of the auger hole. If a coal seam was thicker than the diameter, a significant amount of coal within the seam was left behind, resulting in low recovery. Even with other concerns, such as intersection of adjacent holes due to poor alignment, production from the auger method peaked in the 1970s with a perception of an inexpensive mining method.

In the meantime early CHM concepts evolved and the current CHM systems emerged in the mid-1970s. In 1981, the RSV thin seam miner, also known as the Metec Miner or Dutch Miner, was introduced. The machine first adopted a drum cutterhead used in underground mining, enabling full excavation of coal seams that vary in thickness. Since the early 1980s, the CHM systems have commercially been used in the USA and the developments of new CHM systems continued, including Tramveyor, Archveyor, Addcar system and Superior Highwall Miner. In the 2000s, the Superior Highwall Miner and the Addcar system were two dominant manufacturers for highwall mining system. In 2014, Addcar was acquired by the Australian company, UGM Mining Solutions.

In the USA, the total run of mine production from highwall mining was estimated to be 59 million tonnes in 2003, consisting of approximately 45 million tonnes from CHM and 14 million tonnes from auger mining, which may account for 4% of total coal production in the country (Zipf and Bhatt 2004). Other countries such as South Africa, India, Russia, Colombia and New Zealand also use highwall mining (Schmidt 2015 and Porathur et al., 2013).

**Highwall mining in Australia**

Duncan Fama et al., (2001) summarised the introduction of highwall mining in Australia: In 1989, highwall mining method was first introduced into Australia. As a trial, ten rectangular entries were mined
up to the penetration depth of 30 m at the Moura Mine. In 1991, Callide Mine commenced commercial highwall mining with auger system. German Creek and Oaky Creek Mines also applied auger mining after the operations in Callide mine. In 1993, Oaky Creek Mine commenced operations with CHM system.

Since 1991, the commercial highwall mining operations have been conducted in approximately 40 pits at 17 coal mines (Christensen 2004). The following coal mines are reported to use highwall mining method (Duncan Fama et al, 2001 and Christensen, 2004):

- CHM system: Oaky Creek, Moura, German Creek, Ulan, Collinsville, Yarrabee, Newlands, Charbon, South Blackwater.
- Auger system: Oaky Creek, Moura, German Creek, Callide, Warkworth, Liddell, Charbon, Gregory, South Blackwater, Goonyella, Jellinbah, Wambo, Foxleigh.

In terms of production figures, 20 Mt of coal have been produced from CHM method and 5 Mt of coal from auger method with peak production of between 3 and 4 Mtpa in 1997-98 (Christensen 2004). During the period of widespread application of highwall mining, the mining method proved to be a productive and safe technology (Christensen 2004) while the mining industry experienced at least 7 panel failures and 3 major roof collapses at different coal mines, which impeded mining operations and sometimes caused damage to the equipment (Shen and Duncan Fama 2001). It is estimated that due to increase in coal prices in the early and mid-2000s the use of highwall mining declined in Australia. It is however anticipated that with the recent lower coal prices some of the open cut mines will be uneconomical due to high stripping ratios and highwall mining will be considered again.

GEOTECHNICAL CONSIDERATIONS IN HIGHWALL MINING

Figure 2 shows a typical panel layout for highwall mining. Given that CHM system is the preferred method over auger mining in the industry, only the considerations for CHM systems are considered in this paper. The panel layout incorporates the web pillars, generated by excavation of a series of entries, and the barrier pillars that are left between the panels to ensure the panel stability. The typical geometry indicates that pillar design in highwall mining has a common ground with that in underground mining. Therefore most of the mechanics for underground pillar design also apply to highwall pillar design (Perry et al., 2015). In this context, the key concepts for highwall pillar design involve panel widths, web and barrier pillar widths as well as the pillar width to mining height (w/h) ratio of pillars, which are somewhat similar to underground coal pillar design.

![Typical panel layout for highwall mining](after NSW DPI technical reference CTR-001)

In some areas, however, highwall pillar design has its own distinct features. Due to the relatively shallow overburden depth compared to underground mining, the web pillars in highwall mining are typically
slender (w/h<3) (Mark, 2006). This is demonstrated by a database of the failed Australian CHM pillar cases with the w/h ratios from 0.6 to 1.4 (Hill 2005). This slender type of pillars are known to fail suddenly accompanying catastrophic domino failures, on the contrary to the non-violent pillar squeeze for larger w/h ratio pillars (4<w/h<8) or the strain-hardening behavior for squat pillars (w/h>10) (Mark 2006). In addition, due to its long penetration up to 500 m, the pillars are not square, but rectangular. Thus, this specific geometry should be considered for pillar design in highwall mining.

Another distinct feature of highwall mining is that the mined entries are excavated by the remotely controlled miner and no roof support is installed. Hence, the entries should be fully mined as soon as possible to avoid time dependent deterioration of roof. For example, the 300 m entry is typically developed in 10-12 hours (Guo et al., 2010). If roof falls occur in the highwall entries the mining operations can be hampered severely. The CHM system can tolerate minor roof falls less than 0.1-0.2 m thick while major roof falls greater than 0.3 m thick enable the equipment to be damaged or even buried (Shen and Duncan Fama 2001). In addition, the roof falls cause the adjacent web pillars to increase their pillar height. The heightened pillars in turn reduce the factor of safety (and the w/h ratio) resulting in the decrease in pillar stability (Shen and Duncan Fama 1999). In this respect, the roof competency has been recognised as one of the most important parameters in highwall mining in Australia (Duncan Fama et al., 2001). It is therefore suggested that the roof span stability assessment should be carried out prior to the commencement of highwall mining operations.

The stability of highwall itself is also critical because the operators and mining equipment are located at the base of a highwall. In general, the highwall itself is stable as the highwall is generated with adequate design and good blasting practices (Fan 2015 and Zipf and Mark 2007). However, the highwall can be destabilised (prior to CHM mining) due to various factors including groundwater flow, weathering, vibration etc (Gardner and Wu 2002). Shen (2014) classified highwall instabilities into three categories based on scales as follows:

- Mass instability: deep seated movement of rock mass into the pit, caused by subsidence or major faulting.
- Face instability: failure at the highwall face due to incomplete pre-split blasting or joints parallel to the highwall face.
- Block instability: localised falls of rock blocks or wedges from the highwall face at various sizes.

Hence, highwall stability should be assessed prior to commencement of highwall mining operations, by means of highwall mapping to identify highwall instabilities (Shen 2014). Once any issues are identified, the preventive measures should be taken, such as guiding an appropriate distance from the highwall face, abandoning a hazardous area, re-blasting the wall and monitoring for wall movement (Shen 2014 and Gardner and Wu 2002). In addition, highwall stability should be assessed during and after the highwall mining operations. This is because of the fact that excavation at the bottom of the highwall may have an impact on the highwall stability. Thus, appropriate pillar design especially for the area of the entry is needed to reduce the likelihood of highwall instability (Fan 2015). An adequate design for entire pillar system is critical as the web pillar failure can lead to a catastrophic highwall failure (Zipf and Bhatt 2004).

Water flow into the mined entries enables mining conditions to deteriorate. Duncan Fama et al., (2001) reported a few cases including a case where reduction in pillar strength led to a panel failure, another case where the weakened coal joints led to a roof failure and an incident of sinking miners into the soft floor. The excessive water flow was believed to contribute to all of those failures. Based on the survey on highwall mining performance in the USA, water problem was one of the most critical factors that impeded the full completion of entries as well as rock fall (Zipf and Mark 2005).

Methane content sometimes hinders highwall mining operations. The suggested maximum level for methane content within a seam is 10 m³/tonne (Duncan Fama et al., 2001). To prevent from the explosive gas, the Moura Mine used an automatic shutdown approach with the equipment when...
explosive concentrations were encountered, whilst inert gases such as carbon dioxide, nitrogen and boiler gas were tested to dilute the gas to a certain level (Mossad et al., 2009).

DESIGN PRACTICE FOR HIGHWALL MINING

Panel/pillar stability

There have been significant advances in underground pillar design with empirical approach. With the database from both successful and unsuccessful case histories, empirical formulae have been derived and successfully employed in the mining industry. Of note is that no sufficient historical data was accumulated when CSIRO initiated highwall mining design guidelines for Australian conditions in the 1990s. The UNSW pillar design methodology was developed in 1995, encompassing squat and rectangular pillars. However, the methodology is not recommended to determine the pillar strength for highwall mining as the highwall pillar widths are outside the empirical data regime in the derivation of those strength formulate (Galvin 2010). Considering the factors such as rectangular pillars, pillars with weak or sheared end constraints, asymmetric loading due to dipping seams or varying overburden depth etc, numerical modelling was used to estimate the strength of pillars in highwall mining conditions described above (Duncan Fama et al., 1995).

Numerical modelling was also used to determine the global layout design. The analysis was carried out to calculate overall “layout strength”, which considers the stability of both individual pillars and the whole mining panel (Adhikary and Duncan Fama 2001). The layout strength is defined as the maximum stress level that the layout remains stable, which is determined by gradually increasing the stress until all pillars are yielded through. Then the safety factor, the ratio of the layout strength to the actual estimated cover stress at any given penetration depth, is determined (Duncan Fama et al., 2001).

NSW DPI (2008) proposed the best practices on highwall mining design using the following empirical guidelines:

- When determining the strength of web pillars, this guideline uses the following values that were calculated in the research of Duncan Fama et al., (1999):

<table>
<thead>
<tr>
<th>Situation</th>
<th>Web pillar strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strong coal, strong contacts</td>
<td>6.0</td>
</tr>
<tr>
<td>Strong coal, weak contacts</td>
<td>5.1</td>
</tr>
<tr>
<td>Weak coal, strong contacts</td>
<td>3.6</td>
</tr>
<tr>
<td>Weak coal, weak contacts</td>
<td>3.1</td>
</tr>
</tbody>
</table>

- Pillar load is calculated by tributary area theory. The maximum overburden depth of the layout is taken to determine the load.
- Then, a minimum safety factor of 1.6 is suggested for new highwall mining operations where no guidance system is employed. When a reliable guidance system is employed, a minimum safety factor of 1.3 is suggested as best practice. A minimum web pillar w/h ratio of 1.0 is also suggested.
- To prevent catastrophic web pillar failures, the maximum critical panel width is restricted to the value that is equal to the shallowest depth of overburden. A minimum pillar w/h ratio of 4 is suggested for the barrier pillars.

In 2006, the U.S. National Institute for Occupational Safety and Health (NIOSH) has developed a pillar design tool for highwall mining, ARMPHS-HWM (Analysis of Retreat Mining Pillar Stability - Highwall Mining) (Anon 2006). ARMPHS-HWM used the Mark-Bieniawski formula, which was first developed for rectangular pillars in underground retreat mining (Mark and Chase 1997). The tributary area theory is
used to estimate pillar stress with the maximum overburden depth or weighted average depth, which is defined as the sum of 0.75 × the maximum overburden depth and 0.25 × the minimum overburden depth. Subsequently, the program provides design guidelines with suggested safety factors.

While this empirical approach is widely employed and regarded as an acceptable methodology in the USA, the design resulted from ARMPS-HWM is often compared and verified with numerical modelling. The reason is that the empirical method doesn’t usually take into account the site-specific information. Vandergrift et al., (2004) described an example of using LAMODEL and UDEC to validate the preliminary design from the empirical tool and to carry out stability analysis of the roof and floor. Perry et al., (2015) also presented an utilisation of FLAC3D, which indicated the possibility of over-designed pillars from ARMPS-HWM.

Entry span stability

Duncan Fama et al., (2001) presented the analytical, numerical, and empirical tools that can be used to assess the entry span stability. CSIRO has developed two analytical models, the laminated span failure model (LSFM) and the coal roof failure model (CRFM), to assess the two dominant failure mechanisms, namely (i) span delamination and snap-through for rock roof and (ii) coal beam shear failure mechanisms in case of leaving coal in the roof (Figure 3). These two among other failure mechanisms are applied to the typical Australian mining conditions of laminated roof with low to moderate in situ stresses. The LSFM predicts the probability of roof falls and the average height of the falls whilst the CRFM calculates the safety factor of coal roof. Numerical approaches such as FLAC or UDEC are conducted when the span failure is expected to be caused by two or more different mechanisms, or the geology is complex to adopt the analytical methods.

In terms of empirical tools, the application of Rock Mass Rating (RMR) as an initial assessment tool was reported in Jim Bridger Mine in the USA (Vandergrift et al., 2004). In Australia, the Moura Mine utilised the RMR as well as the Coal Mine Roof Rating (CMRR) (Hoelle 2003) to analyse the roof span stability. They suggested that once the potential span instability is estimated after the initial assessment, the appropriate actions should be taken. One is to leave coal in the roof (Duncan Fama et al., 2001), or cut some of the immediate weak roof with coal to prevent the roof falls (Fan, 2015). Otherwise, an alternative mining system such as auger system can be considered as auger system tolerates most adverse roof conditions (Duncan Fama et al., 2001).

OTHER DESIGN ISSUES

Other relevant design issues, such as highwall mining through old auger holes, multiple seam mining interactions and backfilling are discussed in the literature. It is suggested that backfilling can be a solution to facilitate highwall mining through old auger holes and thick seam mining, which may be of interest to Australian coal miners considering using a highwall mining system.

Highwall mining through old auger holes
In the USA, about 20% of the highwall mining operations recovered the coal in areas where auger mining was already conducted (Zipf and Bhatt 2004). Highwall mining through old auger holes provides additional opportunities to recover as old auger holes were rarely driven up to 60 m and usually no more than 30 m (Amick 2007). The current CHM systems can drive with the average of 300 m and up to 500 m. NIOSH used numerical modelling to estimate the strength of web pillars containing old auger holes, which is generally reduced to at least 15% to 25% compared with the strength of solid web pillars (Zipf and Mark 2005).

When mine design is carried out in these situations, the reduced strength of web pillars containing auger holes leads to the increase in the web pillar width, resulting in low recovery. In this case, backfilling of the previous auger holes may be considered as an alternative. In a case study, the calculated extraction ratio was largely increased to 65% with backfilling method from 40% without filling the old auger holes; the backfilling was successfully tried in a few holes though (Amick 2007).

Multiple seam highwall mining

When thick seams exceed the working height of the highwall mining equipment or a thick seam splits into thinner seams, multiple seam mining approach can be considered (Ross et al., 1999; Zipf and Mark 2005). The ideal situation in multiple seam highwall mining is to accurately stack upper and lower seam pillars through the entire penetration, which is difficult to be carried out without guidance systems on the highwall miner (Zipf and Mark 2007). In the worst case scenario, when the upper pillars are located at the mid-span of underlying interburden, the interburden thickness is of paramount importance (Zipf 2006). If two working seams are not separated sufficiently, the two seams interact and the strength of the two-layer system is less than the individual strength (Zipf and Bhatt 2004). The interburden thickness of less than 4 m, which is approximately equivalent of one highwall mining entry, is regarded as critical (Zipf and Mark 2005). Figure 4 represents the likely pillar-beam failure mechanism in the worst case scenario where the tensile fracture can occur in the lower part of interburden beam (Zipf 2006).

![Figure 4: Pillar-beam failure mechanism (after Zipf 2006)](image)

Even if the pillars are perfectly aligned by accurate stacking through the entire penetration, the “tall pillar failure mechanisms” (i.e., combined thicknesses of the seams) (Figure 5) should be considered (Zipf 2006). In this case, Zipf (2006) suggested a combined height of two seams plus interburden for pillar design due to the possibility of weak interburden failure. Zipf (2006) also mentioned that the strength of those tall pillars is 20% to 30% less compared with the strength of the single upper or lower pillar, which may pose risks of pillar and highwall failures.

![Figure 5: Tall pillar failure mechanism (after Zipf 2006)](image)
Backfilling

Extensive research on the geomechanics of backfill has been carried out over 30 years (Clark and Boyd 1998). However, it had not been widely applied in the Australian coal mining industry due to the perception that backfill was not economically viable at prevailing coal prices (BFP Consultants 1997). In general, the research from BFP Consultants suggested a number of backfill materials, the effectiveness of partial filling and partial recovery rather than complete backfilling and the practical aspects of how to place fill materials (Hume and Searle 1998). The other research from MINSERVE (1999) suggested that in a thick seam, coal recovery may be significantly improved by backfilling. While the recovery rate of a 8 m thick seam highwall mining was to be 30% without backfilling, for example, field trials of two-pass highwall mining with backfilling the lower pass demonstrated that the recovery of up to 64% was achievable when coal seams are greater than 4 m (MINSERVE 1999).

SUMMARY AND CONCLUSION

This paper reviewed the highwall mining experience, including the history of highwall mining from the USA in the 1940s and the introduction into Australia in the 1990s. The geotechnical considerations in highwall mining involving slender and rectangular shape of pillars, unsupported roof span, highwall stability, water and gas considerations are discussed. Design practice for highwall mining is summarised by presenting various approaches for panel/pillar stability and span stability analysis. The relevant design issues for future research topics and challenges such as highwall mining through old auger holes, multiple seam highwall mining and backfilling are also discussed. Some of open cut mines may be able to consider adopting highwall mining as an alternative mining system, on the basis of an understanding of some of the features, the geotechnical considerations and the relevant issues of highwall mining.

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