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On behalf of the organising committee, we welcome you to the 2019 Coal Operators’ Conference (Coal 2019). The conference is hosted by the mining group of the faculty of Engineering and Information Sciences, The University of Wollongong. The conference is supported by The Illawarra Branch of the AusIMM and The Mine Managers Association of Australia. The conference is preceded by a one day workshop on mine dust control, presented by well known Australian dust control experts and practitioners.

Since its inception in 1998, the conference has become a popular venue for well-known industry experts and talented scientists presenting papers of important significance with the aim of making mining operations safer, efficient and environmentally friendly. There is a good balance of papers covering various topics related to new technologies and developments, with diverse topics on ground control, mine gas drainage, coal and rock bursts, mine fires and dust control. In addition to home based papers there are papers from China, Czech Republic, Germany, India, Indonesia, Iran, and the UK.

In this conference 32 papers out a total of 37 papers will be presented over the two day conference, dealing with various issues pertinent to improving the efficiency and safety of mining operation in Australia and beyond. Most papers are related to underground coal mining operations with a limited number being on open cut. A variety of topics, which include geology, mine exploration, mining methods, ground control, gas drainage, gas outburst and rock bursts, mine fires and safety, spontaneous combustion, mine automation, mine management and logistics are discussed in the conference.

Unfortunately, the mining industry, both coal and metal mining, remains affected by difficulties as we still hear about dangers in the mining industry from all round the world. The recent mining disaster in the Czech Republic CSM coal mine near the Polish border with 13 fatalities due to a methane explosion on December 20th and nine fatalities in a Russian Solikamsk potash mine fire, about 1500 km north east of Moscow on December 23rd 2018, are both a sombre reminder of the challenges the industry is still facing. It is the collective responsibility of the mining fraternity to share this information and to strive for future safe mining operations world-wide. It is therefore pleasing to see some papers with a mix of authors from Australia and other countries, demonstrating the spirit of cooperation aiming to solve and overcome the difficult challenges that the industry is facing. We must continue working collectively and pursue vigorously the aim of a safer mining environment as we move forward armed with improved technological endeavours. This conference serves as a vital platform for spreading knowledge and expertise for safer working conditions.

Special thanks to the editorial panel members and the paper reviewers; Libin Gong for typesetting the conference proceeding, Johlene Morrison for the creation of the conference website and its management, Kevin Marston and Shahin Aziz for conference general management and Gypsy Jones café for catering. The University of Wollongong printery staffs Terry Campani for designing the conference proceedings covers page and his colleagues for printing the conference proceedings. Finally, sincere thanks to the 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. All proceedings are available on line through the University of Wollongong Library Research Online http://ro.uow.edu.au/coal

Professor Naj Aziz
Conference executive chairman

Mr Robert J Kininmonth
Conference executive co-chair
CONFERENCE BOOK COVERS
DEVELOPING AND USING EMPIRICAL MODELS FOR GEOTECHNICAL DESIGN IN UNDERGROUND COAL MINING

Mark Colwell

ABSTRACT: This paper addresses several historical and contemporary issues that relate to the various modelling and analysis techniques utilised in the Australian underground coal industry to assist in geotechnical design and in particular ground support design, while focussing on the development and use of empirical techniques, which have substantially contributed to improving safety and productivity both in Australia and overseas.

In the field of mining geotechnics, the potential experience base is huge. For example many longwall panels are mined each year, and each one is a full-scale test of a pillar design and the ground support system(s) employed. The basic approach taken to develop empirical design tools utilising such information is described with examples including Analysis of Longwall Tailgate Serviceability (ALTS), Analysis and Design of Rib Support (ADRS) and Analysis and Design of Faceroad Roof Support (ADFRS).

This paper demonstrates that empirical techniques (based on a sound mechanistic understanding of the geotechnical environment) are particularly relevant and beneficial in dealing with the complexities of geotechnical design associated with underground coal mining resulting in far superior design tools as compared to that offered by numerical or purely analytical techniques.

INTRODUCTION AND BACKGROUND

This paper addresses several historical and contemporary issues that relate to the various modelling and analysis techniques utilised in the Australian underground coal industry to assist in geotechnical design and in particular ground support design.

Furthermore it is hoped that this paper can make a significant contribution (moving forward) that allows geotechnical practitioners to better understand that an empirical method based on a sound mechanistic understanding of the geotechnical environment can readily become a colliery’s principal operational design tool in conjunction with the mine-site geotechnical engineer utilising their experience, training, site specific knowledge as well as simple common sense and other geotechnical tools if required.

There is a view held by a number of geotechnical practitioners (and regurgitated by others), which tends to pigeonhole the use of empirical techniques for “planning purposes” as opposed to being one’s frontline ground support design tool, with others envisioning that empirical techniques will become unnecessary, e.g. Seedsman et al (2009) argue, “that if the mechanics of the problem are understood, the simplicity of the coal mining geometry and modern stress analysis tools makes the empirical approach unnecessary.” It is highly unlikely that this will ever occur or is in anyway desirable for effective ground support design, however developing an analytical model (even at the conceptual stage) to assist in understanding the mechanics of the problem is extremely important.

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Brady and Brown (2004) state, “Whenever possible, it is desirable that mining rock mechanics problems be solved using the analytical tools and engineering mechanics-based approaches discussed in later chapters of this book”. The issue is however that it is not always possible and in fact with respect to underground coal mining is rarely, if ever, fully possible.

In addition even if the analytical model becomes a credible tool for ground support design purposes, then the end result of virtually all analytical models is a Factor of Safety (FOS) and to determine a suitable design FOS (and then to relate this value to a Probability of Failure), requires a database of information which can be statistically analysed if the model is to be applied across an entire industry without further site specific calibration.

In reality all geotechnical models utilised for design associated with underground coal mining are in fact empirical in nature as calibration is typically required and sound engineering judgement will always need to be used when applying any design outcomes. It does not matter whether the engine room of the model is analytical or numerical as either will require significant calibration prior to the model being effectively or confidently utilised for design purposes, whereas the calibration process is intrinsically a part of an empirical model whose engine room is an industry database.

It is acknowledged that empirical methods based on simple statistical analysis (e.g. simple linear regression forcing the regression line through the origin and/or “eyeballing” in upper and lower design boundaries) can be extremely limited in their application, particularly where it has not been demonstrated that the plotted parameters faithfully represent the mechanics of the problem. Geotechnical practitioners who have been exposed to these types of empirical methods may find it difficult to appreciate that empirical techniques, such as ALTS 2009 (Colwell and Frith, 2009), ADRS (Colwell, 2004) and ADFRS (Colwell and Frith, 2012), can actually be based on a sound mechanistic understanding of the geotechnical environment.

Galvin (2016) acknowledges that empirical methods, which are based on a clear understanding of the underlying physical phenomenon, assumptions made and the databases used for their development, can form the bases of valuable design tools. However, Galvin’s (2016) overall commentary with respect to empirical methods is confused and somewhat disparaging particularly in relation to the various statistical techniques that can be used and how they are employed i.e. via the use of spreadsheet software. In addition, as no specific examples are given it is unclear as to who’s work Galvin (2016) is attempting to disparage using terminology such as “oblivious” and “contrived”.

In the development of ALTS, ADRS and ADFRS design methodologies, effective use was made of statistical techniques ranging from mean and standard deviation, simple linear regression, multiple linear regression and logistic regression and in many instances, Microsoft Excel was utilised to undertake the analyses. The selection of the statistical technique to be employed is dependent on a clear understanding of the issues listed by Galvin (2016) as well as a clear understanding of the desired outcome.

Galvin (2016) in reviewing the empirical method states, “Advances in numerical analysis are providing more reliable insight into the mechanistic relationships between parameters and their relative influence on ground behaviour, hence resulting in some applications of empirical analysis becoming obsolete”, while providing no practical examples of what empirical techniques have become “obsolete”. Furthermore, as this paper demonstrates, numerical analysis does not provide a more reliable insight with respect ground/roadway behaviour associated with underground coal mining. Numerical modelling is primarily a stress analysis tool and its application to ground support design is a dubious extension of its application.
Unfortunately there are numerous examples of where geotechnical practitioners overstate the benefits of numerical modelling while virtually deriding the use of empirical techniques in terms of ground support design. In one such instance (as a result of ACARP Project C12011: Review of Barrier Pillar, Bleeder, Chain Pillars in Weak Strata and Thick Coal) in relation to roadway/tailgate strata failure, Fabjanczyk et al (2006) state that, “The conditions required to initiate the failure mechanisms are site specific and make empirical techniques inappropriate” and yet all four sites in that project (i.e. Moranbah North, North Goonyella, Southland/Austar and Angus Place) are a part of the ALTS database and where the tailgate performance was readily predicted by the ALTS Design Methodology at that time (i.e. ALTS II, refer Colwell, et al, 2003).

It would appear that rock mechanics scientists have a strong preference with respect to numerical modelling, however it is the author’s contention that a good engineer will use all that science has to offer to model what is a complex environment to achieve realistic, cost effective and safe design outcomes that can be effectively utilised by a colliery (and if possible by an entire underground coal industry) as a part of their Strata Management Plan (SMP).

To repeat and reference all the misleading comments with respect to empirical techniques would take up much of the word allotment for this paper, however in the context of this paper it is worth reviewing one specific example in relation to rock mass classification systems, which are typically utilised by empirical techniques with respect to ground support design.

It is not uncommon to read in technical papers/publications (e.g. Calleja, 2008, Seedsman, 2008 and Galvin, 2016) a quote (or aspects thereof) taken from one of the editions of Brady and Brown (2004) with respect to rock mass classification systems which reads; “Although the use of this approach is superficially attractive, it has a number of serious shortcomings and must be used only with extreme care. The classification scheme approach does not always fully evaluate important aspects of a problem, so that if blindly applied without any supporting analysis of the mechanics of the problem, it can lead to disastrous results.”

If one reads and accepts the above quote then it is highly likely that one would not be inclined to utilise rock mass classification systems or empirical techniques for ground support design, however directly following the above quote and within the same paragraph, Brady and Brown (2004) state, “It is particularly important to recognise that the classification schemes give reliable results only for the rock masses and circumstances for which the guidelines for their application were originally developed. It is for this reason that considerable success has been achieved in using the approach to interpolate experience within one mine or a group of closely related mines.”

The above quote puts an entirely different perspective on the issue and significantly changes the context, such that a geotechnical practitioner can now better appreciate the benefit of empirical techniques or at least take the time to investigate such a technique for their particular application. It is only reasonable that if the above Brady and Brown (2004) quotes are used, then they should be used in their entirety.

Other branches of science still embrace the empirical approach, for example the empirical method has resulted in many of the phenomenal accomplishments of modern medicine. There is still no satisfactory “numerical model” for the human body, yet new drugs are approved every day based on empirical studies and controlled trials. Medicine does not question the benefits or the continued need of this basic methodology; it simply understands the limitations and applies the results accordingly both in terms of research and medical treatment.

Why therefore would we as geotechnical engineers deny ourselves the use of the same successful scientific method? Furthermore there are three basic questions that an engineer
should ask in applying a design technique; “is the technique credible in terms of the application for which it is being used?”, “what are the practical benefits that the technique offers in terms of safety, productivity and cost-effectiveness?” and lastly, “what benefits, if any, would other methods provide, over and above the technique in question?”

Therefore it is unfortunate but necessary that a paper such as this is published, so that the geotechnical community can better understand how empirical methods based on a sound mechanistic understanding of the geotechnical environment are developed and used as well as the significant benefit they offer the minesite geotechnical engineer.

**MODELLING OF COAL MINE ROOF/RIB BEHAVIOUR**

In relation to coal mine roof/rib behaviour and geotechnical evaluation/ground support design, there have been four basic approaches in relation to either modelling this environment or in the development of design tools which in alphabetical order are:

- Analytical
- Empirical
- Numerical
- Physical

An example of a Physical Model is illustrated in Figure 1, which is the same as Figure 126 taken from Hoek and Brown (1980) where they state, “Figure 126 illustrates the buckling of slabs in the roof and floor of an excavation in a high horizontal stress field. This type of failure was observed in model studies conducted by the Australian Coal Industry Research Laboratory (ACIRL) in an attempt to simulate the structural and stress conditions in the coalfields near Sydney, Australia.”

Brady and Brown (2004) discuss the limitations of physical modelling, however they also state. “The method is particularly appropriate where structural features exercise a dominant role in rock mass response”. In terms of horizontally bedded roof, the major structural feature is the bedding along which delamination (i.e. tensile/shear failure) occurs resulting in thinner (or slender) beams, which can buckle under sufficient horizontal stress with ensuing shear failure of the rock as illustrated by Figure 2. Figure 3 illustrates similar behaviour associated with the ribs where delamination can occur along the cleat, coal joints as well as mining induced fractures.

The physical modelling studies conducted by ACIRL were extremely useful in better understanding the behaviour and failure mechanisms associated with underground coal mine roof and ribs and in conjunction with research undertaken since that time (particularly in reviewing extensometry information), it is clearly apparent that slender beam/column behaviour or buckling is the dominant behavioural/failure mechanism occurring within the immediate coal mine roof and rib which, if not controlled, leads to large scale roof/rib displacement and potentially a major collapse.

Figure 4 is a sonic probe extensometer plot displaying significant roof displacement, i.e. Total Roof Displacement (TRD) of approximately 90 mm and Height of Softening (HOS) to at least 4 m above the roofline. This particular plot clearly illustrates both how the roof delaminates into thinner beams and also how the 1.8 m bolts (that were utilised to reinforce this roof) modify the beam behaviour via the roof reinforcement mechanism of “beam building”. The concept being that the bolts and cables create “thicker” beams within the reinforced section (or the primary bolted interval) and that a thicker beam will have a greater lateral load bearing capacity than a thinner beam.
Figure 1: ACIRL coal mine roadway physical model (after Hoek and Brown, 1980)

Figure 2: Coal mine roadway roof displaying buckling and shear failure due to horizontal stress (after ARBS Help File, 2012)
Figure 3: Blockside ribline buckling – Appin colliery (after Colwell, 2004)

Figure 4 illustrates the behaviour (or response) of a section of maingate roof during and subsequent to longwall retreat. Under the action of horizontal stress, bedding and/or weakness planes can be forced apart and thinner discrete beds or beams of roof material start to form. This inevitably results in discernible roof displacements and roof softening (i.e. delamination) progressing higher into the roof as the magnitude of deformation (i.e. TRD) increases.
The dashed horizontal line on Figure 4 represents the top of the 1.8 m primary bolted interval and there is an obvious difference in roof behaviour at this location within the roof. The response of the roof within the bolted interval is that of thicker beams as compared to the roof material overlying this interval (which is one of a series of thinner beams) up to the extent of the roof softening, which is approximately 4 m.

It is worth emphasising; that in stating that slender beam/column behaviour is the dominant behavioural/failure mechanism occurring within the immediate coal mine roof and rib, in no way should that be interpreted to mean it is the only failure mechanism. In terms of analytical modelling, Colwell and Frith (2010) utilise the mechanics of slender beam behaviour as the basis for the development of AMCMRR (Analytical Model for Coal Mine Roof Reinforcement), while accounting for the compressive failure of thicker beams which do not fail due to buckling.

It is worth noting that ALTS 2009 and AMCMRR were developed via the ALTS 2006 Project, which was conducted over a three year period while being funded directly by individual collieries and mining companies. Midway through the project, Emeritus Professor Ted Brown was commissioned to review the analytical model in its form at that time and concurred that, “under the elevated horizontal stress conditions applying in many underground coal mines, particularly following longwall extraction, slender beam behaviour is, indeed, the dominant coal mine roof mechanism. It follows that this mechanism should be accounted for in any empirical, analytical or numerical approach to underground coal mine roof; and roof support design.” (Brown, 2007).

Brown (2007) provided further guidance and addressed some of the limitations of the analytical model, which were also recognised by the developers and where possible these were addressed prior to providing the final analytical model to the industry i.e. AMCMRR, with the major advancement, being the reasonable quantification of the beam building roof reinforcement mechanism.

The reinforcement mechanism or concept of beam building (as discussed by Mark, 2000) associated with the installation of roof bolts has long been recognised in the underground coal mining industry. While numerous researchers (e.g. Peng 1998, Gale et al 1992 and Seedsman et al 2009) have discussed the various mechanisms by which the bolts act to “create thicker beams” (i.e. by maintaining friction on bedding planes etc.), AMCMRR was the first geotechnical design tool that in a practical way attempted to quantify the beam building effect.

Like all geotechnical models AMCMRR has limitations, however it provides a reasonable analytical solution to a complex issue while realistically incorporating the reinforcement mechanisms associated with slender beam behaviour i.e. beam building and mechanical advantage. Furthermore it was always the intent that AMCMRR should initially be used and calibrated on a site by site basis (in terms of a suitable design FOS). This in fact is typical of how many (if not most) analytical and numerical models are utilised by experienced geotechnical engineers.

In addition, AMCMRR was developed to be used in conjunction with the ALTS Design Methodology as it is the ALTS Design Methodology that has been formulated to complement the Australian minesite risk management approach to strata control/management and this aspect will be discussed later in the paper.

The original ALTS Design methodology (Colwell, 1998) and ALTS II (Colwell et al, 2003) specifically dealt with tailgate design for the vast bulk of tailgates/chain pillars subject to double pass longwall extraction with the focus being the tailgate intersection performance with the retreating longwall face (i.e. refer Position d, Figure 5) as the design condition.
As a result of the ALTS 2006 Project, ALTS 2009 now contains roof support design modules for Maingate Belt (MGB) Road roof support design as well tailgates subject to Single and Super Stress Notch conditions and in addition the ADRS module for rib support assessment and design.

The interested reader is referred to Colwell and Frith (2009) and Colwell (2010a) for a more detailed description of ALTS 2009, however for the purposes of this paper the development of the MGB Roof Support Design Module within ALTS 2009 will be used to illustrate the process by which an empirical model is developed and will be discussed in the subsequent section of this paper.

With the advent of more powerful computers in the late 80’s early 90’s, a significant number of researchers in the field of rock mechanics moved away from empirical, analytical and physical models to numerical modelling. While the modelling of rock behaviour using numerical methods has improved and mathematical routines have been developed in an attempt to account for both elastic and plastic behaviour (e.g. FLAC – Gadde and Peng, 2005, Gale and Tarrant, 1997 and Gale et al, 2004; 3STRESS – Medhurst, 1996 and MAP3D – Palmer and Morrison, 2005), the various models do not incorporate mathematical routines associated with buckling.

In addition these researchers have been considering geometries (or setting up their models) which contain structural elements that, by their very nature, cannot buckle and must fail in either direct compression (as one would observe in a laboratory based strength test) or shear. This is in complete contrast to the slender beams associated with coal mine strata, which either form the immediate roof or quickly develop within the immediate roof due to roadway formation or as a result of a horizontal stress increase.

Therefore it is not surprising that the issue of buckling as a failure mechanism about coal mine openings/roadways has been largely ignored by researchers that rely heavily on numerical modelling in an attempt to replicate and understand roadway behaviour. However Gale (2018) takes this to a whole new level; while accepting that it is common to see coal ribs, which are “apparently” buckled (e.g. refer Figure 3), for Gale (2018), somehow this is caused by conjugate shear failure behind the buckled zone. Gale (2018) applies this same theoretical numerical modelling to the roof as well and illustrates how erroneous conclusions can be reached where the necessary mathematical equations/code are missing.
One of the primary reasons that numerical models (as they are being used with respect to the Australian underground coal industry) require a high level of calibration via parameter manipulation is that the modelling process does not include the mechanistic principles of the dominant behavioural/failure mechanism occurring within the roof/ribs. The geotechnical environment, rock mass failure modes and the way in which roof and rib support interacts with the rock mass are complex issues and therefore it is generally recognised that without prudent simplification, the complexity of the problem will overwhelm all current geotechnical methods of modelling, however to simply ignore and now apparently dismiss the dominant behavioural/failure mechanism occurring within the roof/ribs means the model has little credibility in terms of actual ground support design.

Tarrant (2005) suggests that researchers utilise numerical modelling, to develop a “better understanding” of roadway behaviour. Tarrant (2005) points out that, “Use of such tools is limited by the simplifications required however when used in conjunction with field measurement and observation, the model findings can be tested and a level of confidence in the results defined.”

The use of numerical modelling in the manner described by Tarrant (2005) generally only provides a calibrated (via measurement) model to then be used for site specific prediction or design. Calibrating a numerical model to a limited number of sites does not provide an underground coal industry with a widely applicable and therefore accepted design tool for roadway ground support design. This is particularly the case when the numerical model being calibrated to said roadway behaviour does not incorporate mathematical code associated with buckling. Invariably one finds that in these instances the researcher does not produce a model or design technique that can be readily utilised by industry and typically the numerical model remains within the domain of the researcher for its application.

A perfect example of the above is ACARP Project C12006 entitled, “Standing Support – It’s Time for an Engineered Solution” (Tarrant, 2005). This project was funded on the premise that, “There are currently no methods that provide mine operators with reliable tailgate support design”, (refer ACARP 2002 Project Selection Newsletter). This statement being made even though 1) ALTS was one of only 11 ACARP geomechanics-related projects which received the highest possible rating (in terms of research quality and industry application) based on a review of 52 underground geomechanics-related projects by Emeritus Professor Ted Brown (refer Brown, 2001) and 2) ALTS II (Colwell, et al, 2003) had been available to and utilised by the industry for some two years at that point and was the subject of the information provided in 2001 to ACARP Project C9108 (i.e. Gale and Hebblewhite, 2005).

The final report associated with ACARP Project C12006 contained Section 4 entitled, “Tailgate Support Design Methodology”, which was all of one page in length such that if a colliery actually wanted to use this “methodology” then it would be quite difficult without utilising the services of SCT Operations Pty Ltd. This contrasts to the author’s final ACARP reports in relation to ALTS (Colwell, 1998), ADRS (Colwell, 2004) and ADFRS (Colwell and Frith, 2012) where the design methodology is fully detailed such that an engineer could programme their own design software if they wanted to.
DEVELOPING AN EMPIRICAL MODEL FOR MAINGATE BELT ROAD ROOF SUPPORT DESIGN

In developing an empirical model, the initial approach is to clearly identify what is the desired outcome and then to assess (via a literature review and using one’s own experience) what are the important factors affecting (or significant predictors of) that outcome and most importantly is that the initial concept and the eventual geotechnical design technique developed is consistent with Newton’s Laws, which govern the physical world associated with an underground coal mine.

In terms of an Australian longwall mine’s belt road, the desired outcome is to quantify the level and type of roof support (as well as timing of installation) required to maintain satisfactory roadway conditions during and subsequent to development (i.e. Position a – ‘B’ Heading, refer Figure 5) and up to the maingate belt intersection with the retreating longwall face (i.e. Position b, refer Figure 5).

In terms of any coal mine roadway, it is necessary to take into consideration that it is not just a roof fall that would be considered an unsatisfactory outcome as practical mining considerations require that the roof be maintained with a satisfactory level of stability during longwall retreat so as to minimise any potential negative impact on longwall production, knowing that productivity and safety can be adversely affected by simply excessive roof convergence trapping equipment (e.g. stage loader) or deteriorating roof conditions necessitating the installation of remedial roof support.

It also needs to be recognised that with respect to belt roads the installation of remedial support about the belt is difficult, will inevitably cause production stoppages and is essentially unacceptable, unlike a tailgate where a low or moderate level of remedial support in isolated areas (while of course never desirable) would be likely to have a lesser impact on safety and/or productivity. Therefore a more conservative approach to belt road roof support design (as compared to a tailgate where there is also the option of standing support) is understandable and generally warranted.

However, once again in terms of practical mining considerations, it is important to appreciate that an overly conservative roof support design may result in the belt road roof “hanging up” in the goaf inbye of the longwall supports causing ventilation problems. Ground support design associated with coal mine roadways is far different from a civil construction associated with tunnelling and has unique challenges.

The next part in the development of the model is to start with a simple model or concept, to which one can subsequently add the layers of complexity if or as required. In this instance (and with experience) the assessment was made that the level of support required to maintain satisfactory roadway conditions throughout the mining cycle, would primarily be a function of 1) some measure or index that relates to the structural integrity/lateral strength of the immediate roof and 2) the horizontal stress acting across the roof as a result of roadway formation then subsequently the belt road horizontal stress concentration effect associated with longwall retreat.

The above determination allows the researcher to collect the necessary minesite information for inclusion in the database, for example:

- All information that relates to the tendon support (i.e. bolts and longer cables) installed off the continuous miner during development as well as any additional support installed prior to longwall retreat with this including (but not limited to) the Mine Manager’s Support Rules,
secondary support plans, roof support hardware specifications, Trigger Action Response Plan (TARP) and Strata Management Plan (SMP).

- All related borehole information e.g. geological/geotechnical logging and geomechanical (laboratory and field) testing of borehole core, underground mapping information with respect to structural discontinuities associated with the roof and the use of an appropriate rock mass classification system that can readily utilise this information.

- All available and relevant in situ stress measurement information and the mine layout/geometry (such as longwall retreat direction and depth of cover contour plan) so that the resultant horizontal stress acting perpendicular to roadway development (i.e. \( \sigma_{R-Dev} \), MPa) and subsequently the stress acting across the roof of the belt road adjacent to the intersection with the longwall face during retreat extraction (i.e. \( \sigma_{R-MGB} \), MPa), can be assessed.

In relation to points 1 and 2 and with respect to previous ALTS research, there had been considerable success in using the roof support ratings PRSUP and GRSUP and the Coal Mine Roof Rating (CMRR) rock mass classification index, so naturally it was decided that these indices would be utilised/assessed initially.

The Primary Roof Support (PRSUP) Rating is a measure of the bolting capacity (kN) per square metre of roof normalised to the primary bolted interval and includes all bolt/tendon support that is installed off the continuous miner or mobile bolter as part of development, whereas the Ground Support (GRSUP) Rating incorporates all bolt and longer tendon roof support installed within the roof of a roadway into a single rating, regardless of when the roof support is installed. This includes all roof bolts, longer tendons, cables and trusses.

The GRSUP is calculated in a similar manner to that of the PRSUP; in fact if no additional support is installed within the roof subsequent to that installed off the continuous miner or mobile bolter then GRSUP will equal PRSUP. The interested reader is referred to Colwell and Frith (2009) where the calculation of these roof support ratings is fully detailed.

The CMRR was calculated using both underground and borehole information as outlined by Colwell (2010b) which is fundamentally based on the information provided by Mark and Molinda (2003). The CMRR has now proven itself extensively in both the Australian and United States underground coal industries to be reliable indicator of the structural competence of the bolted mine roof interval. The primary reasons being, 1) the CMRR incorporates an index that directly relates to how a rock/coal unit will delaminate under horizontal stress and the resultant average beam thickness and 2) realistically weights the impact of beam thickness and UCS in terms of the structural integrity of the roof. The statement by Galvin (2016) that, “the CMRR does not take account of behaviour mechanisms” is simply incorrect.

The resultant horizontal stress acting perpendicular to the direction of drivage (i.e. \( \sigma_{R-Dev} \)) is calculated using equation 1 (refer Page 92, Hoek and Brown, 1980) which is derived from Mohr’s Circles:

\[
\sigma_{R-Dev} = \left[0.5 \times (\sigma_{H} + \sigma_{h}) - 0.5 \times (\sigma_{H} - \sigma_{h}) \times \cos(2\beta)\right]
\]

where:
- \( \sigma_{H} \) is the magnitude of the major horizontal stress (MPa);
- \( \sigma_{h} \) is the magnitude of the minor horizontal stress (MPa);
- \( \beta \) is the angle between the roadway direction and the orientation of the major horizontal stress (refer Figure 6).
It should be noted that $\sigma_{R-Dev}$ and $\sigma_{R-MGB}$ are calculated and specifically relate to the primary bolted interval and therefore the Young’s Modulus ($E$, GPa) and Poisson’s Ratio ($\nu$) of the coal/rock units associated with bolted interval are required as the in situ stress measurements are typically conducted in roof units above the bolted interval or below the coal seam.

The change and increase in horizontal stress in the roof that occurs about the belt road intersection with the longwall face during retreat extraction (i.e. refer Position b – Figure 5) is often referred to as Maingate Stress Notching. The magnitude of the resultant stress (in MPa) is denoted as $\sigma_{R-MGB}$ and use was made of the research findings of Gale and Matthews (1992), Mark et al (1998) and Su and Hasenfus (1995) to estimate $\sigma_{R-MGB}$.

Gale and Matthews (1992) discuss the horizontal stress monitoring undertaken as a part of their study and the methods used so as to link a Stress Concentration Factor (SCF) to the angle between the longwall retreat direction and the major horizontal stress direction (i.e. the angle “$\beta$” - refer Figure 6). It is worth noting that Gale and Matthews (1992) detailed certain limitations associated with the methods used, however it is assessed that what was provided (i.e. the SCF relationship to $\beta$) is a reasonable approximation to a complex issue so that in conjunction with the Mark et al (1998) and Su and Hasenfus (1995) information, Colwell and Frith (2009) were able to analytically develop a process by which a reasonable estimate for $\sigma_{R-MGB}$ can be made and the interested reader is referred to Colwell and Frith (2009) where its calculation is fully detailed.

Therefore it is recognised that $\sigma_{R-MGB}$ is an approximation and not an “exact” calculation and therefore while its units are MPa (as used within AMCMRR), within an empirical model $\sigma_{R-MGB}$ is an index that is utilised to “capture an effect”, which in this instance is the stress notching effect about the belt road intersection with the retreating longwall face. Ultimately it is the strength of the various statistical relationships that will determine the quality of these indices.

The type of statistical technique which is used to analyse a database will primarily depend on whether the outcome (referred to as the dependent variable) is continuous or categorical. Many interesting dependent variables/outcomes are categorical e.g. patients may live or die, people may pass or fail exams, coal rib lines may collapse or be stable, roadway roof performance is satisfactory or unsatisfactory and so on. A range of statistical techniques have been developed.
for analysing data with categorical dependent variables, including discriminant analysis, probit analysis, log-linear regression and logistic regression.

The statistical technique of logistic regression was used in the development of ALTS, ADRS and ADFRS when analysing the various databases in terms of criteria based categorical outcomes, where the outcome is typically classified as satisfactory, manageable (being an appropriate category as all Australian coal mines employ a TARP and SMP) and unsatisfactory.

Logistic regression allows for the classification of cases or observations into two (or more) populations based on an outcome. Logistic regression is also able to distinguish which parameters (referred to as the independent variables) are significant predictors of a particular outcome and to then rank and quantify the relative importance of these independent variables on the outcome.

However in developing an empirical method for maingate belt road roof support design, it was decided that the desired outcome is to quantify the level of support to maintain satisfactory roof conditions subsequent to development and during adjacent longwall extraction via the roof support indices PRSUP and GRSUP (and therefore a continuous outcome), with the CMRR, $\sigma_R-\text{Dev}$ and $\sigma_R-MCB$ as the significant predictors of that outcome. In this instance the most appropriate statistical technique to use is multiple linear regressions.

The primary roof support database (developed via the various ALTS research projects) is drawn from both the travel road/tailgate and maingate belt road databases and comprises 109 Cases (representing 38 Collieries; being 36 longwall and two bord and pillar). Figure 7 presents the relationship for PRSUP along “Headings” plotted against the CMRR minus the Strong Bed Adjustment (i.e. CMRR - SBADJ).

![Figure 7: PRSUP v’s (CMRR - SBADJ and $\sigma_R$-Dev) - Headings](image)

It should be noted that “Headings” refers to that section of the belt road, travel road or tailgate (refer Figure 5) between cut-through intersections, while “Intersections” refers to those sections of the gateroad that intersect with the cut-throughs.

It was found during the course of the ALTS research that most collieries (as a part of their Support Rules) increase roof support levels within the intersections and for certain distances either side of the cut-through edge along the heading (i.e. inbye and outbye of the intersections).
This practice is consistent with the both the geotechnical environment and operational factors (as discussed by Colwell and Frith, 2009). Due to space constraints associated with a conference paper it is only the Headings analyses that are presented.

One of the most important concepts incorporated into the CMRR is that of the Strong Bed Adjustment (SBADJ). Many years of experience with roof bolting (in Australia and overseas) has found that the overall structural competence of mine roof is very often determined by the quality of the most competent bed (i.e. rock unit) within the bolted interval.

There are relatively few collieries in Australia (i.e. approximately 10% to 15% of the database) where the SBADJ is a significant issue or component of the CMRR, however it is apparent from the analyses (and field investigations) that at those collieries where the SBADJ is a significant component of the CMRR, if for any reason anchorage within the strong bed is compromised (i.e. due to gloving, water, installation difficulties) or the strong bed is absent within the bolted interval (for example due to a thickening of weaker strata beneath the strong bed or the strong bed “lenses out”) then roof performance can be significantly and adversely effected particularly during longwall retreat.

If the reinforcing benefits associated with anchoring the bolts in a strong bed are lost then the primary roof support level (i.e. PRSUP) becomes even more critical in maintaining sufficient lateral load bearing capacity within the immediate roof. In such cases prudent engineering judgement dictates that the recommended level of primary roof support (as a part of the overall design process) is assessed in terms of the CMRR less the SBADJ.

The exponential trendline as well as the upper and lower boundaries (which statistically represent a 95% confidence level/interval) are displayed on Figure 7. Utilising the exponential trendline, the strength of the correlation ($R^2$) between PRSUP and CMRR - SBADJ for Headings is a relatively high 0.68. However with the inclusion of $\sigma_{R-Dev}$ the correlation increases significantly and is an exceptionally high 0.87.

Figure 7 clearly illustrates that the PRSUP v’s (CMRR - SBADJ) relationships for varying stress levels acting across the roof (i.e. $\sigma_{R-Dev}$) fit seamlessly within the upper and lower boundaries. The maximum $\sigma_{R-Dev}$ associated with the primary roof support database is approximately 22.5 MPa.

The maingate belt road database comprises 58 satisfactory cases representing 33 longwall operations where the CMRR ranges from 25 to approximately 80 and the cover depth ranges from 100m to 510m. As previously discussed, the installation of remedial support about the belt is essentially unacceptable and therefore only those cases with a satisfactory outcome were utilised i.e. there were no production delays or safety concerns attributable to roof instability and certainly no roof falls or remedial roof support measures required.

Figure 8 plots GRSUP against the CMRR and $\sigma_{R-MGB}$ and while a strong relationship exists between simply GRSUP and CMRR (i.e. $R^2 = 0.73$), with the inclusion of $\sigma_{R-MGB}$ the correlation increases significantly and is an extraordinarily high 0.89. Once again the GRSUP v’s CMRR relationships for varying stress levels acting across the roof (i.e. $\sigma_{R-MGB}$) fit seamlessly within the upper and lower boundaries. The maximum $\sigma_{R-MGB}$ associated with the maingate belt road database is approximately 45 MPa.

In many ways there is a great deal of similarity between the basic engineering Factor of Safety equation (i.e. FOS = Load Bearing Capacity of a Structure/Applied Load) and the GRSUP/CMRR/$\sigma_{R-MGB}$ relationship displayed on Figure 8. In terms of a belt road roof adjacent to the intersection with a longwall face, clearly $\sigma_{R-MGB}$ is analogous to the Applied Load. If we replace the term Load Bearing Capacity of a Structure with Load Bearing Capacity of a
**Reinforced Structure**, then GRSUP and CMRR essentially “become” the **Reinforced Roof Structure** (as determined within AMCMRR). The difference between the two equations comes down to the outcome.

Prior to finalising the design methodology for maingate belt road design it is important to recognise that in addition to the geotechnical/risk related issues, operational factors directly influence the level of primary roof support utilised within the gateroads of Australian collieries. For example many collieries elect to install a level of primary roof support off the continuous miner greater than would be required to simply maintain satisfactory roadway conditions during and subsequent to development while prior to longwall retreat, as it may be operationally more convenient or effective to do so off the miner rather than installing secondary support at a later stage to maintain satisfactory roadway conditions during longwall retreat.

To assist in ascertaining this base level of primary roof support (designated as PRSUP_{Dev}) Colwell and Frith (2009) combined the maingate belt road database with a portion of the primary roof support database where it was assessed that the level of roof support installed off the miner was simply to maintain satisfactory roadway conditions associated with development and prior to longwall retreat. Therefore via this combined database the operational issue related to installing a level of roof support greater than that required to effectively deal with the resultant horizontal stress acting across the roof (i.e. $\sigma_{R-MGB}$ as the case may be) is substantially eliminated from the analyses.

This combined database of 90 cases for Headings presented in Figures 9 essentially represents a level of reinforced roof stability in terms of:

- A tolerable level of risk specific to Australian collieries and
- The two principal geotechnical drivers being the structural integrity of the immediate roof (as measured by the CMRR) and the resultant stress ($\sigma_{R}$).
Based on the preceding information, the extraordinary strength of the various relationships associated with Figures 7, 8 and 9 (i.e. $R^2$ values $\approx 0.9$) and previous experience, a maingate belt road roof support design methodology could readily be developed for Australian longwall operations, which provided options with respect to the timing of installation as well as the required level of support at the various stages of the development/longwall extraction cycle.

In developing the ALTS technique and database, information was collected over a 12 year period (i.e. early 1997 to end 2008) representing 36 longwall operations. During the course of the various ALTS and ADRS projects approximately 120 underground inspections were conducted to assist in the formulation of the database. During each site visit, information was collected via the underground inspections and discussions with colliery personnel. Subsequent to an underground inspection, a site inspection report was prepared which was then forwarded to the respective colliery for review and confirmation. This process was undertaken to ensure the integrity of the information contained in the database.

Furthermore contained in the ALTS 2009 software package for longwall gateroad design (which is utilised by approximately 70% of Australian longwall operations) are several resource documents (under the Help Menu) two of which are entitled, “ALTS Summary” and “Computational Aspects of ALTS”. Within these documents the process by which the recommended, upper and lower values (for each of the various design parameters) are determined and utilised is clearly outlined and design examples are provided to demonstrate the use of the equations and design methodology so as to fully detail the inner workings of the software, which is rarely if ever provided by the numerical modeller.

DEVELOPING ROOF SUPPORT PATTERNS UTILISING ALTS 2009

In utilising ALTS 2009 and the roof support indices PRSUP and GRSUP to develop roof support patterns there are three basic issues to be aware of:

- The various design methodologies within ALTS 2009 specifically relate to roof/rib support practices utilised within the Australian underground coal industry both in terms of risk and the hardware/installation practices employed.
• Typically a minesite geotechnical engineer will be starting with some established ground support practices at the mine and where there are changing conditions in terms of the geotechnical environment (e.g. CMRR, geological discontinuities encountered by the continuous miner affecting the CMRR and changes in the in situ stress) initially it is a quantitative measure of the necessary support densities, which are required. While at the feasibility stage it is essentially all about evaluating a quantitative measure of the required support densities, which PRSUP and GRSUP provide.

• ALTS 2009 (design methodologies and software) was formulated to be used by a suitably qualified geotechnical engineer (e.g. RPEQ Geotechnical Mining) with a reasonable level of underground coal mining experience.

BOLT AND CABLE LENGTH SELECTION

ALTS 2009 limits the bolt length selection from 1.5 m to 2.7 m, which is the extent of the database. However with respect to point 1 it is important to note that 62 of 109 primary roof support database cases utilise a 2.1 m length bolt while 34 employ a 1.8 m bolt length where the average is 2.0 m. Therefore the Australian underground coal operations predominantly utilise 2.1 m and 1.8 m bolt lengths and this is a reasonable starting point while then taking into account other issues that would impact on the final roof bolt length selection.

Furthermore ALTS and the subsequent ADFRS research demonstrated that for fully encapsulated bolts (as typically installed in Australia and accepting a certain level of variability in the quality of installation between pits and operators), the significant predictors of the bolts’ effectiveness in terms of roof reinforcement are individually 1) the length of the bolts, 2) the capacity of the bolts (i.e. Typical Ultimate Tensile Strength, kN) and 3) the bolting density as well as when these are combined into the overall PRSUP Rating. The research of Mark et al (2001) reached the same conclusions.

A clear example of an underground coal industry (at that time) demonstrating that bolt length and bolting density has a significant impact on roof stability and the research approach taken, is that associated with the roof support design methodology developed by the CDC - The National Institute for Occupational Safety and Health (NIOSH) for the United States underground coal industry, ARBS (Analysis of Roof Bolt Systems – Mark et al, 2001).

Mark et al (2001) explain that with respect to their research, the “starting point” was an industry that had more than 1,500 roof falls occur each year in U.S. coal mines where the average bolt length was approximately 5 ft (≈ 1.5 m which is the Australian lower limit), four bolts per row had become the near universal standard and bolt spacing was limited by law to 5 ft, but was seldom less than 4 ft. In analysing their database the outcome variable, which measured the success of the roof support system, was the number of Mine Safety and Health Administration (MSHA) reportable roof falls that occurred per 10,000 ft (3,048 m) of drivage, which were solely related to roof falls during or within 18 months of development and excluded any falls associated with longwall retreat.

In analysing their database Mark et al (2001) utilised a form of discriminant analysis based on the following three categories:

- **Failures** (more than 1.5 roof falls per 10,000 ft of drivage)
- **Intermediate** (the roof fall rate is between 0.4 and 1.5 falls per 10,000 ft)
- **Successes** (the roof fall rate is less than 0.4 falls per 10,000 ft)
With the discriminant analyses separating Failures and Successes such that the Intermediate category essentially represented the design condition. Essentially ARBS represents a roof support design technique that hopes to reduce roof fall rates rather than eliminate them.

The above definition/categorisation of roof failure (or level of roof falls) would be totally unacceptable in the Australian underground coal industry. This discussion highlights that a country’s tolerable level of risk is a critical factor in the level (and type of support e.g. bolt length) utilised and in developing a roof support design methodology; and also why the use of multiple linear regression, based solely on the satisfactory cases, was the appropriate approach for the development of a maingate belt road roof support design technique for Australian longwall operations.

With respect to cable length selection, it is strongly recommended that cable length should be determined such that it is suitably anchored in competent upper roof strata and that it exceeds the potential/likely HOS during longwall retreat. Prior colliery experience, the supplier’s recommendations on anchorage length and the use of AMCMRR can assist in this assessment.

**ROOF SUPPORT PATTERNS**

Research has found and it has been explained on numerous occasions during workshops/training courses; if a pit has a reasonable understanding of its geotechnical environment including nature’s variation within that environment and the ability to predict or at least account for such variations, in terms of the data inputs to ALTS 2009, ADFRS and ADRS, then the resultant data outputs will virtually always be consistent and correlate well with the required roof/rib support levels to maintain satisfactory roadway conditions.

The roof support (i.e. PRSUP and GRSUP) ranges provided in ALTS 2009 allow an engineer “to be an engineer” and make judgements. A suitably qualified geotechnical engineer with a reasonable level of underground coal experience and trained in the use of ALTS 2009 should be able to readily make these judgements, such as how to convert the selected/design PRSUP and GRSUP into a suitable balance of bolts and longer cables and where to position the tendons.

For example, if one were to employ a six bolt primary roof support pattern then the positioning and angle of installation of the two outer and two inner bolts is dictated to a large degree by the continuous miner, does one then need to be Einstein to work out where the 5th and 6th bolts should be installed?

Furthermore AMCMRR, which was developed to be utilised to complement ALTS, requires as data input the tendon position across the roadway, its angle of installation, length, pre-load applied and whether it is fully encapsulated, point anchored and/or post-grouted, such that ALTS 2009 and AMCMRR should be used in conjunction with one-another for the best design outcome.

In terms of where in the support ranges one selects as the design PRSUP and GRSUP; during the ALTS training courses numerous and simple commonsense examples are provided e.g. achieving poor encapsulation – move higher in the range, high level of confidence in CMRR selected – stay near the recommended, the pit considers they have the best and most conscientious development crews in Australia which is supported/confirmed by routine audits (e.g. pull tests) – possibly move lower in range, the continuous miners employed by the pit can’t close up the cut-out distance from the face to last line of support sufficiently when encountering significant roof movement on development – move higher in the range, the colliery has in place real time extensometry monitoring as well as a properly considered TARP that is
conscientiously adhered to – potentially move lower in the range and the list of issues to consider goes on.

The ALTS database intrinsically deals with all these issues and the design recommendations emanating from ALTS correlate with manageable levels of risk that have been assessed in terms of an Australian database and the geotechnical engineer using ALTS 2009 for maingate belt road roof support design can take great comfort in the fact that approximately 90% of the reason(s) for the level of roof support required is accounted for by reasonable estimates for the CMRR and the horizontal stress acting across the roof.

CONCLUSIONS

The idea portrayed by some that empirical modelling per se and the resultant statistical relationships are simplistic and are limited in their application is at best misguided and at worst, misleading. Quality empirical modelling i.e. empirical modelling based on a sound mechanistic understanding of the geotechnical environment; is in fact a scientific process of significant challenge and complexity.

With respect to the underground coal geotechnical environment; empirical modelling allows for the development of practical and fully engineered design methodologies and techniques/tools that can provide an entire industry and the minesite strata control engineer with timely solutions to complex geotechnical design issues. The development and success of ALTS 2009, ADRS and ADFRS for geotechnical design are also consistent with the thoughts of Hustrulid (2006) where he indicates that marked progress in the field of mining rock mechanics requires, “the careful collection, analysis and presentation of field/mine experience.”

In relation to empirical modelling Salamon (1989) states, “The main advantage of this approach is its firm links to actual experience. Thus, if it is judiciously applied, it can hardly result in a totally wrong answer. Also, in our legalistic world, it has the added advantage of defensibility in a court of law. After all, it is based on actual happenings and is not just a figment of imagination”. ALTS 2009, ADRS and ADFRS go even further, as the statistical relationships and the way they are utilised as a part of the design methodologies intrinsically represent a tolerable level of risk specific to Australian collieries, which a numerical modelling approach (as employed in the Australian underground coal industry) is simply not capable of doing.

While being a strong advocate for the advancement of numerical analysis, Salamon (1989) gave a reasonable and professionally balanced view when discussing both numerical and empirical techniques and was of the belief that mathematical modelling (not numerical as Galvin, 2016 indicates) is essential in the field of strata control. Of course getting the mathematical modelling/equations correct is crucial within many branches of science such as spaceflight trajectory/analysis. If the mathematics is wrong or necessary mathematical equations/code are missing, then the programmed computer model can be worthless or even dangerous leading to disastrous results.

As indicated in the introduction, it is the world of medicine and medical/pharmaceutical research and their approach in dealing with nature that has had a significant impact on the author’s approach to the natural environment associated with strata control. The interested reader is referred to the Australian Clinical Trials website (www.australianclinicaltrials.gov.au) where under “Your stories” there are many interesting stories from patients, researchers and medical practitioners concerning the benefits of clinical trials. However probably the most poignant statement is that of researcher Robyn Ward, then Professor of Medicine and Director of the Cancer Centre at Prince of Wales Hospital and University of New South Wales, “Knowing what works and what doesn’t is fundamental to the evidence base that underpins medicine. Without it, we are just snake oil salesmen.”
Professor Ward’s statement of course applies to all branches of science and while not using the same colourful language similar sentiments are expressed by Hustrulid (2006) and Salamon (1989) with respect to the need for real world evidence and justification for the development and use of geotechnical design techniques.

REFERENCES

Colwell, M G, 2010a. Pillar design procedures and research methodologies - can there or should there be a unified approach?, in Proceedings of the 2nd Australasian Underground Coal Control in Mining Conference: Ground Control in Mining: Technology and Practice, 23 - 24 November 2010, pp 67 - 77 (Sydney).


THE LIMITATIONS AND POTENTIAL DESIGN RISKS WHEN APPLYING EMPIRICALLY-DERIVED COAL PILLAR STRENGTH EQUATIONS TO REAL-LIFE MINE STABILITY PROBLEMS

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ABSTRACT: Determining coal pillar strength equations from databases of stable and/or failed case-histories is more than 50 years old and has been applied in different countries by different researchers in a range of mining situations. Whilst common wisdom sensibly limits the use of the resultant pillar strength equations and methods to design scenarios that are consistent with the founding database, there are a number of examples whereby failures have occurred as a direct result of applying empirical design methods to coal pillar design problems that are inconsistent with the founding database.

The paper explores the reasons as to why empirically-derived coal pillar strength equations tend to be problem-specific, and so should perhaps be considered as providing no more than a pillar strength “index”. These include the non-consideration of overburden horizontal stress within the mine stability problem, an inadequate definition of super-critical overburden behaviour as it applies to standing coal pillars and the non-consideration of overburden displacement and coal pillar strain limits, all of which combine to potentially complicate and so confuse the back-analysis of coal pillar strength from failed cases.

A modified coal pillar design representation and model is presented based on coal pillars acting to reinforce a horizontally-stressed overburden, rather than suspend an otherwise unstable self-loaded overburden or section thereof, the latter having been at the core of historical empirical studies into coal pillar strength and stability.

INTRODUCTION

The inspiration for this paper is founded in three statements from two eminent persons in the field of coal pillar strength research, Jim Galvin and Essie Esterhuizen, as follows:

“Both Salamon and Munro and UNSW based the derivation of their pillar strength formulae on a criteria that the diameter of a panel of pillars, W, had to at least equal the depth of mining, H. This was thought to result in full tributary loading. It is now known that there are some mining environments included in both the South African and Australian databases in which mining span must exceed depth by a considerable margin in order to achieve full deadweight loading. Hence, it is logical to conclude that these data points may have contributed to pillar strength being overestimated by Salamon and Munro and UNSW (underline added by authors). Normally, this should be of no consequence because it is reflected in the probability of design success associated with any given safety factor.” Galvin 2006

“Empirical models, based on the analysis of large numbers of case histories, have found wide acceptance as a tool for engineering design. The application of empirical models is...”

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limited by the restriction that they should not be used beyond the limits of the empirical base from which they were developed.” Esterhuizen 2014

“Pillar design criteria based on field experience have met with mixed success” – Galvin 2016

This paper addresses three inevitable but currently unresolved questions that follow from these statements:

(i) How significant might the overestimate of pillar strength be when the varying contribution of the overburden to the stability or instability of mine workings supported with coal pillars is ignored when back-analysing failed cases?

(ii) How important and what are the various “limits” that need to be considered when determining whether an empirical method can be used with confidence or not?

(iii) Based on (i) and (ii), in hindsight is it reasonable for empirically-derived pillar strength equations to use units (e.g. psi or MPa)?

A CONCEPTUAL CAUSE AND EFFECT MODEL FOR OVERBURDEN STABILITY/INSTABILITY

If the influence of the overburden above collapsed mine workings supported by coal pillars is to be quantified, a conceptual model is required that incorporates all of the primary controls. Such a model was first outlined in Frith and Reed 2017 which is developed further herein to produce a Ground Reaction Curve (GRC) representation that incorporates the associated principles.

A series of simple thought-experiments relating to super-critical extractions, as stated by Galvin 2006 as W/H > 1, can demonstrate the direct influence of key geotechnical parameters on overburden stability or instability.

Figure 1 represents a massive sandstone with no vertical joints or horizontal stress. Under this scenario, it is obvious that the overburden will remain stable across a wide extraction span, with the associated GRC rapidly reducing to zero stress (i.e. a self-supporting condition).

Figure 2 is identical to Figure 1, but contains laminated material. Under this scenario, the overburden would initially “sag” downwards and eventually exceed a “critical” level of movement marking the onset of instability to full overburden collapse to the surface (i.e. Tributary Area Loading in coal pillar design terminology).
Figure 2: Thought-experiment and ground curve: laminated overburden with no vertical joints or horizontal stress

Figure 3 contains the overburden representations of Figures 1 and 2, but introduces vertical joints and horizontal stress. Vertical jointing is omnipresent in coal measures strata sequences and is characterised by zero cohesion and a friction angle that varies according to surface conditions along the joint. Therefore, vertical shear resistance cannot develop along the joint without a normal confining stress, which is horizontal in this case. Therefore irrespective of overburden lithology, without the presence of horizontal stress the presence of vertical jointing will inevitably result in a GRC as shown in Figure 4, namely an unstable overburden with zero stiffness from the outset.

Figure 3: Thought-experiment representations: massive and laminated overburdens including vertical joints and horizontal stress

Figure 4: GRC- vertical joints present without horizontal stress
Once overburden horizontal stress is included, the influence of overburden lithology and vertical shear resistance along vertical joints can be combined, resulting in a range of possible GRC outcomes as generally illustrated in Figure 5.

![Figure 5: Varying influence of overburden lithology, vertical joints and horizontal stress on GRC\'s](image)

For any given span between either barriers or solid coal, overburden stability or instability is directly influenced by the presence or absence of massive strata units, and the horizontal stress generating a stabilising influence along vertical joints. In combination, these two geotechnical parameters logically give rise to two distinctly different mechanisms associated with overburden instability and eventual collapse:

- delamination and/or associated sag whilst ever vertical joints remain horizontally “clamped”, or
- a “plug” type collapse due to vertical joints becoming unstable from insufficient horizontal confinement.

The first point of developing this conceptual model is to justify the concept that overburden conditions dictate the importance of the coal pillars in maintaining stable mine workings, not the other way round.

**MEASUREMENT DATA THAT SUPPORTS THE BASIS OF THE CONCEPTUAL OVERBURDEN MODEL**

At the 36th International Conference on Ground Control in Mining, (ICGCM), concern was raised that without field-data substantiating the magnitude of the overburden stress drop before collapse was initiated in a GRC (as per Figure 5), there was no way of quantifying the extent by which the nature of the overburden may have influenced back-analysed coal pillar strengths from failed cases. It was also strongly suggested that the overburden GRC may commonly approximate to that shown in Figure 4, hence there was no significant disconnect between back-analysed and actual coal pillar strengths.

These statements are hypothetically correct and even the fundamental work of Esterhuizen et al 2010 provides only limited assistance in addressing them as the study was undertaken without field data validating the numerically produced GRC's (see Figure 6), the statement being made that “it is difficult to measure the ground response curve in actual underground excavations because of the significant loads that would have to be applied to balance the original ground pressure”. This again is correct, but it doesn't alter the need for field data to validate GRCs if their value in the design of underground mines using coal pillars is to be maximised.
A literature search has been undertaken to identify surface subsidence data from above coal pillar systems that have allowed overburden collapses. Fortunately, such subsidence data exists from the Lake Macquarie area of NSW in Australia relating to partial extraction layouts, and despite pillar system failure relating to floor rather than pillar failure, as it is the GRC of the overburden that is of specific interest, this makes no material difference.

Ditton and Sutherland 2013 describe a significant unexpected subsidence exceedance related to a completed Duncan Method of partial extraction workings in the Fassifern Seam with a soft claystone floor at the Tasman Mine. The exceedance was within the 3 North Panel and was limited to an area whereby the standard five heading development layout was extended to six headings over a panel length of four pillars (i.e. approximately 180 m) as shown in Figure 7.

Figure 6: Ground response curves at the centre of a 300 m wide panel in weak and strong overburden strata at 150 m and 450 m depth of cover (Esterhuizen et al 2010)

Figure 7: Modified Duncan panels (2b-North, 3 North and 4 South) to west of transition zone and cover contours (Ditton and Sutherland 2013)
Figure 8 shows the variation in vertical subsidence across the exceedance area of six headings and the adjacent area of five headings, Figure 9 containing a time-dependent plot of $S_{\text{max}}$ above the six heading area as estimated from Figure 8. Based on these two figures and what is known about the Tasman geotechnical environment (including from McTyer and Sutherland 2011), the following comments are made:

(i) Immediately post-mining, the development of $S_{\text{max}}$ follows a trend of a high rate that decelerates to a longer-term condition, in this case very slow creep at an average rate of 3 mm/month.

(ii) Based on extrapolation, once $S_{\text{max}}$ reached 150 mm to 200 mm, an obvious change of state occurred via a substantial increase in settlement rate (to around 50 mm/month) followed by a deceleration back to a second longer-term condition with $S_{\text{max}} > 450$ mm (NB questions-marks have been included in Figure 9 as reading frequencies do not allow the exact nature of the transition to be reliably identified).

(iii) The floor of the mine workings was soft and heaved following mining (Figure 10), coal pillars remaining relatively intact.

(iv) The overburden included the Teralba Conglomerate in the order of 20 m thick.

(v) The subsidence exceedance was restricted to the six headings area with an extraction width between solid coal of 248 m, as compared to the five heading area at a width of 203 m as shown in Figure 8.

(vi) Both five and six heading areas were at a cover depth in the order of 130 m (see Figure 8) and so using the selection criterion used by Salamon and Munro as well as UNSW, both were under full tributary area loading to surface and presumably had identical pillar loadings.

(vii) The increased subsidence did not extend into adjacent five heading areas, indicating that the “event” was controlled by the excavation geometry and overburden stability, rather than pillar stability.

(viii) With a seam outcrop around three sides of the mine shown in Figure 11, tectonic horizontal stress levels were low, as discussed by McTyer and Sutherland 2011.

The conclusion reached is that the increased subsidence in the six heading area was likely to have been caused by the onset of “plug” type overburden instability within the Teralba Conglomerate once a critical or threshold level of overburden movement had been reached (around 150 mm) due to ongoing overburden creep with time due to floor instability.
Figure 8: Delayed subsidence above 3 North in level 1 area (Ditton and Sutherland 2013)

Figure 9: Time dependent variation in $S_{\text{max}}$ within 3 North subsidence exceedance area

Figure 10: Moderate floor heave in 3-North panel (outbye)

Figure 11: North-south section across the tasman mine lease showing the fassifern seam outcrop around the sugarloaf range (H:V = 20:1) - Fassifern Seam is shown in black (McTyer and Sutherland 2011)
Four similar examples to that described for the 3 North Panel at Tasman Mine have been found in the published literature.

Figure 12: An example of the rate of development and magnitude of vertical surface displacement due to bearing capacity failure of the floor in partial extraction workings at a depth of 160 m (Galvin 2016)

Figure 13: Surface settlements from Newvale colliery (Shirley and Fagg 2017)

Figure 14: Subsidence vs time plot for 5NE panel, Newvale 2 colliery (Vasundhara et al 1998)
Figure 15: Remnant mine layout, NE panels, Newvale 2 colliery (Vasundhara et al 1998)

Figure 16: Some of the known rapid floor heave and subsidence events in the great northern seam at Awaba colliery (Seedsman 2008)

Figure 12 is taken from Galvin 2016 and Figure 13 from Shirley and Fagg 2017 relating to the section of the Newvale Mine associated with the well-publicised lowering of the lake foreshore in Chain Valley Bay in the late 1980’s/early 1990’s. Figure 14 contains measured surface settlements vs time above the 5NE Panel at Newvale 2 Colliery (Vasundhara et al 1998) with Figure 15 showing the remnant mining layout in the NE Panels. Figure 16 is taken from Seedsman 2008 whereby he back-analysed what were described as “rapid floor heave and
“subsidence events” at Awaba Colliery, noting that some of these events occurred at depths as low as 60 m with coal pillars on 20 m centres having pillar Factor of Safety (FoS) values in excess of 3.

All of the listed case histories have a number of common characteristics that are of particular relevance herein:

(i) The partially extracted areas are all super-critical in that W/H values are >> 1. The partial extraction areas shown in Figures 12 and 15 are in the order of 600 m x 600 m with cover depths in the order of 160 m to 180 m, resulting in W/H values in the order of 3 to 4.

(ii) Following the completion of partial extraction, initial subsidence levels were low with longer-term trends consisting of very low rates of creep with time (where sufficient measurements were taken to allow time-dependent trends to be reliably identified).

(iii) At subsidence values of 150 mm and greater, the rate of subsidence with time rapidly increased with longer-term subsidence levels approaching 1 m.

(iv) Longer-term settlement rates commonly returned to low values with no evidence of further sudden increases, even several years later.

(v) In Great Northern Seam workings, the presence of thick, massive conglomerate units in the overburden as well as soft tuffaceous floor material, can be reliably inferred.

Figure 17 (Mills and Edwards 1997) succinctly summarised some 29 remnant pillar case histories incorporating soft floor measures in the Lake Macquarie area, whereby measured surface subsidence magnitudes clearly polarise into two distinctly different categories, namely “pillars intact” for $S_{max}/T < 0.075$ and “pillars failed” with $S_{max}/T$ varying as a direct function of pillar w/h. A gradual transition between the two conditions via increasing surface subsidence over time is also identified in four of the case histories. With the stated seam thickness involved being 2.2m to 2.5 m, provides for an upper $S_{max}$ value for “pillars intact” (or overburden stable in more general terms) in the order of 165 mm to 187.5 mm.

![Subsidence as function of pillar width to height ratio for soft floor failures](Mills and Edwards 1997)

Figure 17: Subsidence as function of pillar width to height ratio for soft floor failures (Mills and Edwards 1997)

Using these specific case histories and relevant geotechnical characteristics, a technical discussion on mechanistic causation can be developed.
The majority of the investigative work that accompanied these delayed and unexpected surface settlement events above partial extraction areas, focused on the failure and compression of soft floor material beneath remnant coal pillars. This is fully understandable given the nature of the floor material and commonly observed floor heave in the workings.

The technical issue that received almost no attention was determining the cause of the rapid increase in the rate of subsidence months after the completion of mining, this being a common feature of the case histories.

Seedsman 2008 makes the statement that “massive conglomerates may span and delay evidence of imminent over-loading of pillars – temporarily stiff loading system”. However when the mined-out areas have widths and lengths in the order of 600 m at cover depths < 200 m, conventional subsidence thinking would inevitably eliminate the possibility that even thick massive conglomerates could span across such areas, this being consistent with the periodic weighting classification for longwalls (Frith and McKavanagh 2000) whereby a 30 m thick conglomerate can only span across an extraction width of 200 m (see Figure 18), certainly not 600 m. Clearly there is a major problem or disconnect using overburden caving behaviour from total extraction panels such as longwalls, to estimate overburden spanning ability when substantial coal pillars are left in place.

![Figure 18: Near-seam, massive strata weighting classification (Frith and McKavanagh 2000)]

The suggested solution to this conundrum, which is as per that stated by Van de Merwe 2006 and addressed in Frith and Reed 2017 in relation to the non-collapsed area at Coalbrook, is found in two aspects of bord and pillar workings and/or partial pillar extraction that are absent from longwall extraction once full caving has been established, namely:

(i) the contribution of the remnant coal pillars to overburden stability, and

(ii) the stabilising influence of horizontal stress within the overburden.

In other words, the GRC for the overburden must be being altered by remnant coal pillars, this being consistent with the idea that coal pillars “reinforce” rather than “suspend” the overburden, as was discussed in Frith and Reed 2017.
As illustrated in Figure 5, it is postulated that the onset of super-critical overburden conditions to surface is dictated by the overburden exceeding a critical level of subsidence, which based on data published by Mills and O’Grady 1998 and Ditton and Frith 2003 was suggested as being in the order of 200 mm by Frith and Reed 2017. Surface extensometry data published by Salamon et al 1972 further indicates the onset of rapid subsidence for values exceeding 150 mm to 200 mm (see Figure 19). The close correlation with measured subsidence magnitudes at the onset of rapid subsidence in the previously described partial extraction cases from the Lake Macquarie area of NSW, is also noted.

Re-defining the role of coal pillars as reinforcing rather than suspending the overburden, was discussed in length by Frith and Reed 2017, a general arrangement for the pillar design problem including a more detailed overburden representation containing thick massive strata units, vertical joints and in situ horizontal stresses, being shown in Figure 20. It is noted and accepted in this representation, that the in situ vertical stresses acting on the production pillars cannot be re-distributed out to any flanking barrier pillars by the action of the overburden, as this would require said pillars to vertically expand or extend due to mining. Therefore, it is only the in situ vertical stress that is released by the formation of mining excavations that can be re-distributed, either in full or more likely, in part.
The controlling influence of overburden horizontal stress on surface subsidence was recognised by Mills 2012, who concluded that “sag” subsidence increases in the presence of high horizontal stress, based on subsidence data from the Newcastle Coalfield in NSW. The explanation for this phenomenon is beyond the scope of this paper, but inevitably leads into the significance of high horizontal stress in roadway roof control as being the driver for uncontrolled buckling of the roof strata, the control and limiting of which is a reinforcing rather than suspension roof support design problem.

If overburden instability to surface (i.e. the “vertical shear slip” line within the GRC in Figure 5 occurs as a distinct change due to vertical joints becoming unstable at subsidence values of 150 mm and greater, then as discussed in Frith and Reed 2017 any coal pillars that subsequently collapse as a direct consequence will have inevitably exceeded their peak strength well prior to this occurring. This is illustrated schematically in Figure 21 and is consistent with the quotation from Galvin 2006, the question then being whether the magnitude and design significance of coal pillar strength over-estimates can be quantified or not?

**HOW SIGNIFICANT CAN COAL PILLAR STRENGTH OVER-ESTIMATES ACTUALLY BE?**

A general equation for the reinforcement of a mine roadway roof was provided in UNSW 2010 and has been slightly modified as follows:

\[
\text{FoS} = \frac{f(P_{\text{roof}}, P_{\text{support}})}{\text{applied load}} \tag{1}
\]

where:
- \( \text{FoS} \) = a measure of stability;
- \( P_{\text{roof}} \) = contribution to stability from the roof strata itself (e.g. Coal Mine Roof Rating);
- \( P_{\text{support}} \) = contribution to stability from installed roof support (e.g. PRSUP);
- applied load = horizontal stress in the case of roadway roof reinforcement.

This basic equation manifests in the statistically significant empirical relationships published by Colwell and Frith 2009 and 2012 relating to primary roof support design in normal width and wider coal mine roadways respectively. It is also the foundation of the AMCRRR Method as published by Colwell and Frith 2010.

Equation 1 can be modified for coal pillar design as follows:

\[
\text{FoS} = \frac{f(P_{\text{overburden}}, P_{\text{pillar}})}{\text{applied load}} \tag{2}
\]

where:
- \( \text{FoS} \) = a measure of stability;
P_{overburden} = stability contribution from the overburden (linked to both the structural competence of the overburden and horizontal stresses acting as outlined previously);

P_{pillar} = stability contribution from coal pillars left in place;

applied load = either horizontal stress or vertical stress based on the problem being reinforcement of suspension respectively (NB P_{overburden} = zero represents the special case of full-tributary area loading to surface with the overburden being critically unstable).

If the model for overburden and coal pillar stability (Figure 20) is accepted along with both the generic GRC concept (Figure 5) and Equation 2, it logically follows that the accuracy of back-analysed coal pillar strengths from failed cases must improve as the stabilising influence of horizontal stress and/or massive strata within the overburden decreases, the overburden GRC then tending towards that shown in Figure 4 with no stabilising contribution from the overburden from the outset.

Two obvious scenarios whereby the stabilising influence of horizontal stress is likely to be minimised or reduced are:

1. Highwall Mining (HWM) from an open cut highwall, this entire issue being at the centre of a recently published pillar strength equation specifically for Australian HWM pillars (Mo et al. 2017), and

2. Decreasing cover depth more generally whereby surface topography effects, the depth of weathering and reducing strata stiffness should increasingly act to reduce horizontal stress magnitudes in the overburden.

The significant mismatch between underground mining failed cases and those from HWM, in Australia at least, is obvious in Figure 22 (Hill 2005). This representation of failed pillar cases resulted in a strong response (Galvin 2006) arguing that as pillar w/h was included in both axes, it was mechanistically incorrect or “ill-advisable” to represent failed cases in this manner. Unfortunately, the more obvious question went either unnoticed or unaddressed at that time, namely why is the stability of HWM pillars so different to those from the underground mining environment?

![Figure 22: Database of pillar collapses – width to height ratio vs. FoS (Hill 2005)](image)

Various arguments have been made by others concerning the weakening influence of minor geological structures on the strength of low w/h ratio pillars due to the absence of pillar confinement, but this would surely equally apply to both HWM and underground coal pillars.
Figure 22 does not support such a hypothesis, particularly given that the number of underground failed cases is greater than those from HWM, such that the effect should be more, rather than less, obvious in the underground failed cases if it were present in low w/h ratio pillars. Nonetheless, UNSW undertook a back-analysis of the HWM failed cases shown in Figure 22 and developed a HWM-specific strength equation (Equation 3 from Canbulat et al 2016) with the equation itself included in Mo et al 2017:

\[ \sigma_p = 4.66(0.56 + 0.44 \text{ w/h}) \text{ or } 2.61 + 2.05 \text{ w/h} \] (3)

However Equation 3 for HWM pillars results in the illogical situation whereby the strength of a long strip pillar (as given by Equation 3) is less than for a square pillar of the same width and height as given by the Salamon and Munro 1967 equation for w/h < 5 as an example (Equation 4).

\[ \sigma_p = 7.176(w^{0.46}/h^{0.66}) \] (4)

However, it is intriguing to consider that Equation 3 is very similar in magnitude to the Bieniawski 1968 equation that was derived from the in situ testing of coal pillars in South Africa as given by Equation 5.

\[ \sigma_p = 2.76 + 1.52 \text{ w/h} \] (5)

The suggestion that coal pillar design in underground mining is a reinforcing problem whereby the stabilising contribution of the overburden needs to be given due consideration, is seemingly strengthened by the fact that Australian HWM coal pillars, where the stabilising influence of overburden horizontal stress is inevitably lower than in underground mining, return lower inferred pillar strengths as compared to pillars in underground mining.

In terms of the stabilising influence of horizontal stress being reduced by ever-decreasing cover depth, it is judged that both the US and Australian collapsed pillar databases are too small to provide any meaningful insights. However, the South African collapsed pillar database in its entirety (i.e. including as many failed cases have been able to identified) is sufficiently large and does show intriguing trends as will now be detailed.
Figure 23 shows the 27 collapsed cases that were used by Salamon and Munro 1967 whereby the FoS using the Salamon and Munro 1967 strength equation (Equation 4) is plotted against cover depth. The collapsed cases centre around an FoS in the order of 1 with no obvious significant trend of increasing FoS with reducing cover depth.

Figure 24 contains those collapsed cases used by Salamon and Munro 1967 and the additional 17 cases considered by Bernard Madden between 1967 and 1988 (Madden 1991). Again the cases centre around 1, but there is a hint of increasing SF with decreasing cover depth at less than 50 m.

Figure 25 includes all of the known collapsed cases from South Africa, with both the Salamon and Munro and Madden data points being differentiated from the other cases. The clear and obvious trend for maximum FoS value to incrementally increase with decreasing cover depth below 100 m being obvious. This is not dissimilar to that shown in Figure 22 for Australian HWM cases, the point being that the South African cases in Figure 25 are all from underground mines. Therefore, perhaps it is not HWM that is the driver for reduced coal pillar strength in HWM, but something far more fundamental?
An explanation for the apparent conundrum of these very high FoS collapsed cases from South Africa was linked to the time-dependent spalling or scaling of pillars, such that pillar collapse must have eventually occurred when the FoS reduced to a suitably low level as compared to the as-formed pillars. Van der Merwe 1993 examined this issue for Vaal Basin collapsed cases and determined that the rate of pillar scaling was a direct function of mining height. Salamon et al 1998 developed the idea that pillar scaling was driven by the swelling of montmorillonite clays within the coal seam, this basic model then being utilised by Canbulat 2010 in his treatment of the time to failure problem (Figure 26).

Figure 26: Design safety factor vs time interval (Canbulat 2010)

Unfortunately the originators of this hypothesis only ever put forward the idea of swelling clays driving pillar scaling as a possible mechanism to explain high Safety Factor (SF) collapsed cases, making the following clarification:

“No direct evidence appears to exist to substantiate the proposed model of pillar scaling. Thus, it is not possible to prove convincingly the validity or otherwise of the approach” (Salamon et al 1998).

They also state that

“It is important that the cause of abnormal collapses be investigated and explained as soon as possible. Such study is likely to find that anomalous behaviour is due to more than one cause. Van der Merwe’s observations imply that pillar scaling could be a reason for some of the premature failures. This deduction and the promising performance of the model proposed here provide a powerful basis for recommending that further study should be initiated to clarify the role of scaling or spalling in pillar mechanics”.

In other words, the work of Salamon et al 1998 was no more than an initial attempt at explaining failed cases with anomalously high FoS values.

One possibility that was not considered, but would be well supported by the overburden reinforcing model for coal pillars outlined herein, is that the high FoS values of failed cases were erroneous in the first instance due to the strength equation that was used substantially overestimating actual coal pillar strength due to the fundamental assumptions used as part of its derivation. When this possibility is combined with the stabilising contribution of the overburden decreasing with decreasing cover depth, the occurrence of pillar scaling over time makes good sense, such scaling being related to under-designed coal pillars that are being compressed over time above yield and towards their ultimate strength, until overburden instability and inevitably pillar collapse eventually occurs.
The suggestion that pillar scaling over time could be due to coal pillars being under-designed to start with is hardly controversial. However, if high pillar FoS values were accepted as correct due to reliance being placed on the coal pillar strength equation that was used in their derivation, then searching for an alternate explanation that was independent of coal pillar strength would be a logical path to follow.

OVERALL SUMMARY

A set of technical arguments backed-up with various field data have been presented to support the hypothesis that bord and pillar—type coal pillar design in underground coal mining is generally one of the coal pillars reinforcing the overburden, such that they combine with the overburden to stabilise the mine workings. The reinforcing representation put forward is comparable with that which is well established in coal mine roadway roof control and contains the same basic input parameters, including the significant influence of horizontal stress magnitudes.

If it is assumed (for the sake of illustration only) that the full-tributary area loading model is correct and is applied within a GRC representation (as shown in Figure 27), then for collapsed pillar cases one must conclude that the actual peak pillar strength is inevitably less than full-tributary area pillar loading, otherwise the two curves would have intersected and system stability would have been returned. In other words, the assumption of full-tributary area loading when back-analysing collapsed pillar cases must result in pillar strength equations that over-estimate the true strength of the pillars by some amount, even in those situations whereby there is no contribution to system stability from the overburden. The pertinent question therefore is the potential magnitude and significance of over-estimated or optimistic coal pillar strengths being returned due to this irresolvable and undefined error, this being the main subject of on-going work.

![Figure 27: Schematic illustration of pillar strength over-estimate even if full-tributary area loading is appropriate in a collapsed case](image)

The reinforcing model logically suggests that pillar strength equations derived from the back-analysis of failed pillar cases under the assumption of full-tributary area loading, in reality more likely provide a measure of the combined stabilising influence of both coal pillars and the overburden. The result of this is that under specific conditions whereby the stabilising influence of horizontal stress within the overburden is either low or indeed absent, highly optimistic mine layouts can inadvertently and unknowingly be developed and implemented if those same pillar strength equations are applied without this realisation.

The design risks associated with determining pillar strengths from back-analysed case histories, particularly from single or only a small number of cases, are potentially substantial if one
accepts this conclusion, the worst-case consequence of which is perhaps best summarised in
the following statement in relation to the Crandall Canyon disaster:

“In a December 3, 2007 submission to MSHA, a consultant explained that the “coal
strength was calibrated from three mining stages in the south panel of Section 36. The
c coal strength was incrementally increased from 900 psi to 1640 psi until modelling results
were consistent with actual conditions. The average cover depth in this calibration
panel was about 1,700 ft. We were told that all the pillars during retreat mining were
stable and only limited yielding occurred at some pillar ribs.” – taken from US Senate
2008.

Therefore, perhaps it might be prudent for coal pillar strength equations that are determined
from case histories to never be given units (such as MPa or psi), even if the associated
statistical correlations are compelling. Treating them as “index” parameters that are linked to a
particular design method, or re-naming them as an “estimate of the combined stabilising
influence of coal pillars and the overburden” for example, may assist in ensuring that such
equations are not used out of context.

Based on the content and conclusions of this paper, the quotation from Esterhuizen 2014 in
regards to only using empirically design methods within the limits of their supporting database,
has great substance, but leaves one further key question unanswered – what are those limits
and how are they defined? Given the critical importance of this to the mine designer, the
developers of the various bord and pillar-type coal pillar design methods may wish to consider
this question in more detail according to the known specific characteristics of the supporting
case histories.

REFERENCES

Bieniawski Z T. 1968. The Effect of Specimen Size on Compressive Strength of Coal.
International Journal of Rock Mechanics and Mining Sciences and Geomechanical
Canbulat, I. 2010. Life of Coal Pillars and Design Considerations. Proc. 2nd Ground Control
Conference, pp 57-66 (Sydney)
and Geotechnical Considerations in Backfilling (The University of New South Wales:
Sydney)
Proceedings of The Coal Operators Conference, pp 73-83 (University of Wollongong).
Report to ACARP, Project C19008.
Fracturing on Groundwater. Final Project Report. Brisbane, Queensland: ACARP Project
C10023.
Ditton, S. Sutherland, T. 2013. Management of Subsidence at the Tasman and Abel Mines -
Issues and Outcomes. Proceedings of COAL 2013, pp. 86-98 (University of Wollongong)
Esterhuizen, G.S. Mark, C. Murphy, M. 2010. The Ground Response Curve and Its Impact on
Pillar Loading in Coal Mines. Proceedings 3rd International Workshop on Coal Pillar
Mechanics and Design. pp 123-131 (Morgantown: West Virginia)
Esterhuizen, G.S. 2014. Extending Empirical Evidence through Numerical Modelling in Rock
Engineering Design. The Journal of The Southern African Institute of Mining and
Metallurgy, Vol. 114, October.


A REVIEW OF THE MECHANICS OF PILLAR BEHAVIOUR

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ABSTRACT: In recent years, the drive to reduce the impacts of surface subsidence has led to mine layout designs that rely for their effectiveness on the long-term stability of pillar systems. This paper reviews the mechanics of coal strength behaviour inferred from laboratory testing of coal specimens as a context to better understand the appropriateness of different pillar design strategies. Laboratory testing of coal specimens to very high confining pressures (163 MPa) illustrates the independence of the two fundamental components of coal strength: cohesive strength and frictional strength. Testing of numerous coal samples from the same coal seam and coal samples from different coal seams illustrate the variability of cohesive strength. The significant influence of frictional strength when confining pressure is available is also apparent. These two fundamental components of coal strength combine to influence the range of pillar behaviours observed in practice. This paper explores the characteristics of these two components and their implications for the application of various pillar design approaches.

INTRODUCTION

In recent years, the drive to reduce the impacts of surface subsidence has led to mine layout designs in New South Wales and Queensland that rely for their effectiveness on the long-term stability of pillar systems. The University of New South Wales (UNSW) pillar design methodology has become a benchmark for assessing long-term stability of pillars in Australia. The method is being applied in a wide range of geological settings and for a broad range of pillar geometries. Galvin, et al. (1999) warn that the UNSW methodology approach is empirical and only suitable for the conditions in which the methodology was developed; a warning that tends to be ignored.

The UNSW approach and most other empirical approaches do not specifically consider the changing characteristics of coal strength or the influence of roof and floor strata on the ability of pillars to develop confinement. This paper describes how two independent components of coal strength combine to give the strength characteristics of coal pillars observed in practice and the implications for pillar design.

The results presented in this paper draw upon a significant body of work that was completed in the 1990’s as part of a collaborative AMIRA project (Gale and Mills 1994) and during a study of pillar behaviour in claystone strata in the Southern Lake Macquarie area (Mills and Edwards 1997). As part of the AMIRA project, a program of testing coal, including at very high confining pressures, showed that coal strength can be characterised as comprising two components, a cohesive component and a frictional component.

The in situ cohesive strength component of Australian coals is estimated to be about 6 MPa (equivalent to the background vertical stress at overburden depths of approximately 240 m) based on observations of the onset of rib spall as overburden depth increases. Mobilisation of the in situ frictional strength component of coal is found to be highly dependent on factors

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external to the coal seam such as geological conditions and the strength of strata units surrounding the coal seam.

**COAL STRENGTH CHARACTERISTICS**

Figure 1 illustrates the load-deformation characteristics of specimens of Pittsburgh Coal loaded over a range of confining pressures. The diagram is reproduced from Kripakov (1981) with original work by Sture (1974) and modified to include metric units.

There are four stages in the deformation history evident at each confining pressure:

1. a linear increase in load in the initial pre-failure stage
2. a peak load when the intact strength is reached
3. a gradual loss of strength with post-peak deformation
4. a steady load or residual strength that is maintained largely independent of deformation.

These four stages of strength development are typical of coal and many other types of rock.

![Figure 1: Stress / Strain behaviour - Pittsburgh Coal (After Kripakov 1981)](image)

The stress-strain (or load-deformation) characteristics of these Pittsburgh Coal specimens show that:
- when the coal fails, it loses strength, a process referred to as brittle failure
- at zero confining pressure, initial intact strength is lost when the coal is loaded beyond its peak strength and failure occurs
- both the peak strength and the residual strength after failure increase significantly with confining pressure

Figure 2 shows the relationship between intact strength and confining pressure and residual strength and confining pressure for Bulli Seam coal with both axes plotted at the same scale. The significant characteristics of these envelopes are:

- the intact and residual strength envelopes are slightly curved, but approximately parallel to each other
- both intact and residual coal strength are very sensitive to applied confining pressure increasing at a rate approximately four times the applied confining pressure
- intact strength is relatively modest by comparison with the strength able to be developed by applying confining pressure.

To investigate coal strength behaviour at high confining pressures, five specimens of Bulli coal were tested in triaxial compression at confining pressures up to 163 MPa. The results, shown in Figure 3, indicate that coal strength behaviour at high confining pressures is consistent with the strength behaviour at lower confining pressures.

- The strength of coal continues to increase with confining pressure at a similar rate for confining pressures up to at least 163 MPa (the maximum tested).
- Coal continues to deform in a brittle fashion (i.e. loses strength after failure) even at high confining pressures.
- The residual strength envelope remains parallel with the intact strength envelope indicating that the strength loss that occurs with failure is constant and independent of confining pressure.
- Stress-strain characteristics are similar at each confining pressure. In each test, there is a well-defined residual strength plateau and a similar level of post-failure deformation required to reach this plateau.

The behaviour observed for Bulli coal in this series of tests is significantly different to the behaviour reported at high confining pressures for coal (Barron and Penn 1992) and some other rock types (Hoek and Brown 1980 and Mogi 1966).

The observation that the intact and residual strength envelopes for coal remain parallel even to very high confining pressures suggests a model of coal strength behaviour involving two independent components. One component that is lost when the coal fails and a second component that is dependent only on confining pressure and is present both before and after coal failure.
If the failure envelopes were linear, these two components would be represented by a Mohr-Coulomb type strength relationship:

\[ \sigma_1 = \sigma_c + \sigma_3 \tan \beta \]  
\[ \sigma_{1r} = \sigma_3 \tan \beta \]  

where \( \sigma_1 \) is the intact strength, \( \sigma_{1r} \) is the residual strength, \( \sigma_c \) is a measure of the internal cohesion within the coal fabric, \( \sigma_3 \) is the confining pressure, and \( \tan \beta \) a scalar of the confining pressure representing the internal frictional forces generated by the confining pressure, whether the coal is broken or unbroken.

When the confining pressure is zero, the internal frictional forces are zero and the unconfined strength of the coal is then equal to \( \sigma_c \).
Although the failure envelopes determined for coal are slightly curved, the concept of the strength relationship based on two components is nevertheless still valid.

This two component model of coal strength behaviour is helpful for understanding the behaviour of coal pillars in the field, the relationships between the behaviour of small and large pillars, and the potential for geological settings to significantly influence coal pillar behaviour.

For convenience, the cohesive component of strength is referred to as cohesive strength and the frictional component of strength is referred to as frictional strength. The characteristics of these two components are described below.

**Characteristics of Cohesive Strength**

Cohesive strength can be envisaged as being associated with the natural bonds that hold the intact coal fabric together. Cohesive strength is the strength that exists when a sample of coal is unconfined. The sample has shape and holds together as an intact material that has
strength. When this sample is overloaded though, the sample loses its original form and becomes a collection of smaller disconnected pieces. In effect, the cohesive strength has gone.

The cohesive component of coal strength is recognised to be:

- associated with the intactness of the coal
- lost when the coal becomes overloaded and fails
- effectively independent of confining pressure
- variable depending on the nature of the coal and factors such as jointing
- a function of test specimen size
- significant to pillar behaviour at low confining pressures
- suitable to be characterised using statistical methods.

The results of laboratory tests shown in Figures 1-3 indicate that cohesive strength is available only until the coal reaches its peak strength. Once it has been overloaded, the cohesive component of strength is lost and cannot be recovered.

The parallel intact and residual strength envelopes shown in Figures 2 and 3 confirm that cohesive strength is independent of confining pressure. Cohesive strength can be thought of as a separate component of material strength that is independent of confining pressure.

**Characteristics of Frictional Strength**

Frictional strength in coal can be envisaged as being a function of confining pressure and the frictional properties of coal. Frictional strength is effectively zero at zero confining pressure but increases with confining pressure at a rate approximately four times the confining pressure.

As described in this section, the frictional component of coal strength is recognised to be:

- independent of cohesive strength
- unaffected by failure or loss of cohesive strength i.e. the same before and after failure because the frictional properties of coal do not change with failure
- insensitive to factors such as deformation, specimen size and coal seam
- significant for larger coal pillar behaviour.

The parallel intact and residual strength envelopes shown in Figures 2 and 3 indicate that frictional strength does not change when cohesive strength is lost i.e. intact frictional strength is effectively the same as residual frictional strength and frictional strength and cohesive strength are independent of each other.

The load deformation plots shown in Figures 1 and 3 indicate a plateau in the residual strength at large deformations indicating that the frictional strength is largely independent of deformation. Frictional strength is present before the coal fails, after it has failed and continues to be present even after large amounts of subsequent deformation. The material has not changed and so the frictional strength has not changed.

Laboratory testing of different coal types, presented in the following section, indicates that the frictional strength of coal varies only slightly between coal seams and this variation is much less than the variation in cohesive strength.
There is limited data available to confirm the relationship between frictional strength and specimen size. Further work is required to confirm the absence of a size effect. The nature of frictional strength suggests there is unlikely to be any size effect so as a first approximation no size effect is assumed.

VARIABILITY OF COAL STRENGTH PROPERTIES

The variability in coal seam strength properties are investigated in this section. Comparison of the strength behaviour of different coal seams indicates that most of the coal strength variability between and within seams comes from the cohesive component. The frictional component varies somewhat between seams but is much more consistent especially within any given coal seam or region and much more dependent on the geological setting in which coal pillars are located.

Specimen Size

The sizes of coal and rock specimens are widely recognised to have a significant effect on unconfined strength. Small laboratory sized specimens tend to be stronger than larger field sized specimens. There are two commonly held explanations for this phenomenon:

- Jointing and other weaknesses reflected in large samples tend to be absent in smaller laboratory sized samples, especially samples selected for testing, because of sampling bias.
- Larger specimens store greater amounts of energy when loaded and this energy promotes the propagation of the unstable microcracks leading to failure at lower loading levels.

Both these factors play a role in the coal strength observed in the field being significantly lower than coal strengths measured on laboratory sized specimens. Hustrulid (1976) summarises empirical techniques that have been used to infer the field strength of coal specimens from laboratory size specimens. In general, he concludes that these relationships can be well represented by a relationship of the form:

\[
\sigma_c = k / \sqrt{D} \quad \text{for } D < 1 \text{ m} \quad (3)
\]

\[
\sigma_c = k \quad \text{for } D > 1 \text{ m} \quad (4)
\]

where \(\sigma_c\) is the field strength, \(k\) is a constant for each coal that relates to its unconfined laboratory strength and \(D\) is the height of an equivalent cubic specimen. The 1m specimen size represents a critical size above which strength is not thought to be reduced any further.

The natural variability of cohesive strength is expected given the association of cohesive strength with the natural cement binding the coal fabric together. Imperfections in the coal structure from one sample to the next inevitably lead to variations in cohesive strength.

Laboratory testing of different coals indicates that cohesive strength varies between and within coal seams. Variations in cleat spacing, the proportion of brighter coal fractions and the natural processes of coalification have potential to lead to variations in the cohesive strength of coal.

Strength Property Variation for Different Coal Seams

The confined strength properties of coals from multiple sites were tested as part of the AMIRA Project (Gale and Mills 1994). The results are shown in Figure 4, together with a selection of coal strength data available from the USA and UK. To allow ready comparison between groups, the lower bound of the intact strength envelope developed for Bulli Seam coal is shown on each of the plots of intact strength and the residual strength for Bulli Seam coal is shown on each of the plots of residual strength.
The strength data presented in Figure 4 is derived primarily from multi-stage triaxial compression tests. The axial strength of the cylindrical test specimens is plotted on the vertical axis. The confining pressure is plotted on the horizontal axis. Unconfined tests on coal show a high degree of variability. Triaxial compression testing shows less variability and is considered a better estimate of cohesive strength. The variability in cohesive strength observed from triaxial testing is similar to the variability in cohesive strength of in situ coal inferred from the failure of small pillars.

The variability of the intact strength is an indication of the variability of the cohesive strength of the coal samples tested. The plots of residual strength provide an indication of the frictional strength of the various coal samples tested.

In Figure 4, the coal strength envelopes from different sites have been grouped together by region and in groups that show similar behaviour.

Bulli Seam coal and Wongawilli Seam coal from the Southern Coalfield, Katoomba Seam coal from the Western Coalfield, and Kupakupa Seam coal from Huntly West Mine, New Zealand, show confined strength properties that are similar (Figure 4a). These coals have generally lower intact confined strength than coals from other regions. The residual strength is less variable than the intact strength.

Coals from the Newcastle Coalfield - Greta, Great Northern, Wallarah - and from Ulan at the northern end of the Western Coalfield show intact confined strength properties that are significantly stronger than Bulli coal (Figure 4b). Greta Seam coal is the strongest coal from this region. The others group into a narrower band that is stronger than Bulli Seam coal, and towards the bottom end of the Greta Seam coal strength envelope.

The residual strength of all the coals in this group are stronger than the residual strength of Bulli Seam coal. The spread of residual strengths is slightly greater than the Bulli group, but much less than the spread of the intact strengths. The residual strength of Greta Seam coal is within the residual strength envelope of the other coals, despite Greta Seam coal having significantly higher intact strength.
Figure 4: Strenth properties of various coal types
Coals from Central Queensland – Castor Seam, German Creek Seam, Harrow Creek Seam - have confined intact strength properties that are greater than Bulli Seam coal but less than the cohesive strength of coal from the Newcastle region. Their frictional strengths residual strength shows a similar relationship to the residual strengths of coals from the other regions.

The strength properties of selected coals from elsewhere in the world are shown in Figure 4d. Coal from Pittsburgh and from the United Kingdom show intact and residual strength properties that are stronger and increase more quickly with confining pressure than the residual strength of Bulli Seam coal. Coal from the Barnsley Hard Seam appears to be an exception which, although significantly stronger than Bulli Seam coal in its intact state, has similar residual strength properties.

**Unconfined Strength Variability from a Single Seam**

Unconfined laboratory tests provide a direct measure of the cohesive strength of coal. Laboratory measurements indicate that unconfined strength of coal is highly variable, not only between different coal types but also between different specimens of the same coal. As an example, Figure 5 shows a summary of 58 laboratory measurements of the unconfined strength of coal samples from the Wongawilli Seam in New South Wales. The strengths measured range from 3 MPa to 22 MPa.

![Figure 5: Variation in Wongawilli Seam coal UCS](image)

This variability is thought to be at least partly associated with the high proportion of bright bands present in Wongawilli Seam coal in the lower part of the seam and the duller coal with a higher mudstone fraction in the upper parts of the seam. With such large variability, it is difficult to be confident from laboratory testing of the unconfined field strength that would be available for pillar design purposes. Some of this variability may be a result of the variation of sample location within the coal seam. The upper part of the Wongawilli Seam has a higher proportion of dull coal and higher ash content. The cohesive strength from this section of the seam is typically higher.

**Variability of cohesive strength under confinement**

The variability in cohesive strength is evident in the triaxial strength results presented in Figure 2 and 4 for a range of coal seams. The variability in cohesive strength evident in these tests is much less than the variability shown in Figure 5 for unconfined tests in the Wongawilli Seam.
The cohesive strength indicated by triaxial testing varies from approximately 10 MPa to approximately 40 MPa. The variability of ±15 MPa is consistent with a variability of 60% of the average unconfined strength of 25 MPa. This observation indicates that the cohesive strength of coal varies approximately ±60% of the average unconfined strength.

**Strength variability in the field**

Wagner (1974) describes a full scale field test to determine the strength of a small pillar. However, field measurement of the variability of cohesive strength in full scale pillars is difficult and expensive. An indication of coal strength variability can be obtained by back analysis of coal pillar stability in areas of small pillars. Salamon and Munro (1967) back analysed 98 stable and 27 collapsed pillar geometries in South Africa. They concluded that to account for strength variability of small pillars, it is necessary to have a factor of safety of greater than 1.6.

A factor of safety of 1.6 represents an anticipated strength variability in small pillars of ±60%. This variability range is consistent with the variability in unconfined coal strength shown in Figure 2 based on the results of triaxial compression tests. The variability in coal strength indicated by the unconfined tests presented in Figure 5 suggests that the variability in laboratory estimates of cohesive strength may be higher than the variability observed in the field.

The variability of the frictional strength properties of coal in the field is difficult to determine with confidence but any natural variation in the properties of coal is considered likely to be small compared to the variability associated with different geological strata units within the coal pillar system more generally. The properties of low strength bedding planes for instance have a significant influence on the strength of the coal pillar systems because they influence the ability of the pillar system to develop confinement. The variability of the pillar strength component associated with friction is much more likely to be a function of changes in geology and strength of the surrounding host strata than a function of any variability of the frictional strength of the coal itself.

**PILLAR STRENGTH BEHAVIOUR**

In this section, the behaviour of pillars of different sizes in different geological settings is considered in the context of the two coal strength characteristics identified from laboratory testing.

**Small width to height ratio pillars**

Small pillars with a width to height ratio of less than about three are recognised to have a geometry that prevents the development of any significant confinement. Pillar strength for these small pillars is controlled largely by the cohesive strength of the coal and the characteristics of this cohesive strength. In practice, the in situ cohesive strength of coal is observed from the onset of rib spall at increasing overburden depth to be approximately 6 MPa for most Australian coals.

Cohesive strength is recognised to be lost relatively suddenly once coal is overloaded. Small width to height ratio pillars that depend on the cohesive strength of coal are therefore prone to sudden loss of strength if they become overcharged.

Cohesive strength is recognised to be variable. The strength of small width to height ratio pillars is therefore also variable. This variation is managed in pillar design using a so called “factor of safety” (or its equivalence as a probability). The intent of this approach is to have enough margin on the low side of the best estimate of coal strength that any variability in cohesive strength is not enough to cause small pillars to become overloaded.
The concept of providing enough margin is particularly important in the design of small pillars because loss of cohesive strength can occur suddenly. In a large panel of similarly sized pillars, there is potential for the failure of one pillar to cause other adjacent pillars to become overloaded. The instability of one pillar can then lead to the collapse of an entire panel with implications for safety underground and on the surface. The hazards include sudden loss of working room underground, windblast and gas expulsion and sudden changes of the ground level on the surface.

A larger factor of safety is applied when the consequences of a collapse or the timeframe over which a collapse would be intolerable are greater. A factor of safety of 1.6 is typically applied for short term stability and 2.1 for longer term stability.

The acceptability or otherwise of factors of safety against these types of events is ultimately a matter of judgement. Consideration in choosing an appropriate factor of safety or probability of failure should take account of the confidence with which the loading characteristics of the site are known, the coal strength variability is understood, and the consequences of any collapse at some time in the future. The factor of safety used in a generalised empirical formula developed primarily from back analysis of small pillars and by implication cohesive coal strength is not necessarily appropriate for a larger pillar system that relies for its strength on frictional strength of the coal and the confinement able to be generated by the surrounding strata.

**Large width to height ratio pillars**

When large width to height ratio pillars become heavily loaded, the coal on the fringes of the pillar is unconfined and becomes overloaded, in the same way that coal in small pillars fails when they become overloaded. When this unconfined coal is overloaded, its cohesive strength is lost causing the coal to fail and rib spall to occur. In a large pillar however, the failure of the rib coal does not mean the pillar system becomes overloaded and loses strength. The failed rib coal instead provides confinement to the pillar edge coal and this confinement increases the strength of the remaining coal in the core of the pillar.

Failure of the rib coal continues deeper into the pillar until the confinement provided by already failed coal, and any other support, generates enough frictional strength in the remaining core of the pillar to support the load on the pillar. A stable equilibrium is then established and pillar edge coal failure stops progressing further into the rib.

External factors such as the presence of a longwall goaf next to a chain pillar or backfill within a roadway significantly increase the rate of confinement provided to the coal rib.

The frictional strength of the coal increases at a rate of about four times the confinement, so a small amount of external confinement leads to a significant increase in frictional strength and hence pillar strength. For coal ribs to be able to generate confinement within the core of a pillar and mobilise the frictional strength of the coal, the strata surrounding the pillar needs to be able to generate an equal and opposite force. This equal and opposite force is typically transferred as shear on bedding and other horizontal planes above and below the pillar.

When the host rock is strong and the roof and floor contacts with the coal pillar are strong, the surrounding strata is typically able to resist the outward shear forces generated in the coal allowing high levels of confinement to be generated within the pillar to mobilise large frictional forces creating very strong pillars. For large pillars in strong roof and floor strata, the confinement provided to the fringe of the pillar by failed coal means the pillar continues to gain strength as it deforms. For these pillars, the upper limit of pillar strength is much greater than the load able to be distributed onto the pillar by the overlying strata.
Figure 6 illustrates the load deformation characteristics of small, medium and large pillars. In large width to height ratio pillars (greater than about 8) and strong roof and floor conditions, pillar strength increases as the pillar is loaded. There is no single point at which the pillar reaches a maximum load i.e. reached a load that could be regarded as the strength of the pillar. Coal on the edges fail (as illustrated by the red shading) but the confinement that this failed coal provides to the central core is more than compensated for by the increased frictional strength of intact coal in the confined core (yellow shading).

Pillar behaviour is significantly different when the pillars are small, the host rock is not strong enough to allow confinement to be developed, or there are low-strength units between the host rock and the coal or within the host strata above or below the coal. Frictional strength of the coal is not able to be fully developed and the strength of the coal pillar is reliant instead on the coal’s cohesive strength. In these circumstances, a large pillar may have small pillar strength characteristics.

A particularly significant effect of low strength roof and floor strata is that once the cohesive strength of the coal is overcome, a large pillar in low strength roof and floor strata can be prone to closing up in much the same way as a small pillar does. This may occur suddenly, but more typically large pillars converge slowly in a process extending over some days to weeks and referred to as a pillar creep.
The failure of pillars is commonly observed on the surface as a subsidence event. For small pillars, a subsidence event is clear evidence of pillar strength being reached and through back calculation, this strength can be estimated. The observation of such a subsidence event above large pillars can also be taken as evidence of their failure. However, for such a failure to occur in large pillars, there must be conditions of low strength roof and floor conditions present.

In strong roof and floor conditions, large pillars would continue to gain strength and could not have failed. Subsidence events above large pillars should therefore be used with caution for estimating the strength of large pillars in strong roof and floor conditions.

**IMPLICATIONS FOR PILLAR DESIGN**

The recognition that coal strength has two components, a cohesive component and a frictional component each with different characteristics, provides a basis to better understand the strengths and limitations of various pillar design approaches.

Small pillars and large pillars in low strength roof and floor conditions rely primarily on the cohesive component of coal strength. The natural variability of cohesive strength and the ease with which it can be determined from back calculation of pillar failures means that statistical analysis of empirical experience is relatively well suited to providing estimates of pillar strength and pillar stability. The concept of a factor of safety to provide a buffer against natural variability in cohesive strength has a credible basis. The factor of safety can be varied to suit the probability of failure considered acceptable for the circumstances.

The concept of a factor of safety to represent statistical variability is less useful for larger pillars that rely for their strength on the frictional component of coal strength. This frictional strength component does not vary to the same degree as cohesive strength and the development of frictional strength depends on external factors such as geological setting and the strength of the surrounding rock strata. These factors are not random variables that can be characterised by statistics. They are certainly not governed by the same statistical processes that are suited to characterising the natural variability of the cohesive component of coal strength.

A different range of criteria become relevant to the design of large pillars. For instance, a chain pillar designed to protect the gateroads of a longwall panel may be technically stable from a pillar strength perspective, but this stability is irrelevant if the adjacent gateroads are not serviceable because of poor roadway conditions induced by excessive loads on the chain pillar. The estimation of maximum design loading for large pillars is typically not governed by considerations of pillar strength or collapse potential but rather by the serviceability of adjacent roadways.

The design of main heading pillars is another example where pillar strength considerations are not the primary concern. Main heading pillars in strong roof and floor conditions can be designed to be individually stable on development or even under the abutment loading from adjacent longwall panels. However, there have been several examples in the Southern Coalfield where low strength horizons in the roof and floor strata that were not able to be detected in advance became mobilised and restricted the frictional strength able to be developed in the core of the pillar despite the roof and floor strata being otherwise strong.

The presence of these low strength horizons and the stress level at which they became mobilised is difficult to detect in advance of mining and even after convergence starts it is not easy to determine where the primary low strength shear horizons are located. Once low strength bedding plane horizons do become mobilised, it is usually too late to prevent a large
scale creep event. Such an event has potential to compromise the serviceability of the main headings and the mine more generally as it has done on several occasions.

One approach to managing the possibility of low strength roof and floor conditions becoming mobilised is to arrange the main heading pillars so that convergence is limited should a creep develop. Four or five headings separated by a much larger central pillar provides a layout that is significantly more robust for convergence control than a large panel of similarly sized pillars.

Characterisation of the strata conditions and the use of numerical modelling and field monitoring provides a pathway for estimating the behaviour of pillars with large width to height ratios that is more credible than reliance on statistical analysis of empirical experience. Statistical experience is readily available for small pillars because small pillars tend to collapse when they become overloaded. It is more difficult to get reliable estimates of strength for large pillars when failure is not defined by a reduction in strength but rather by other criteria such as the serviceability of adjacent roadways. Cassie and Mills (1992) describe the application of field measurements as the basis for a numerical modelling assessment of the behaviour of large pillars, in this case in low strength roof and floor conditions.

Intermediate size pillars with width to height ratios in the range four to six rely for their strength on both the cohesive strength of coal and the frictional strength of coal. For these pillars there is a transition in behaviour. The design of pillars in this range requires an understanding of both the cohesive strength characteristics of coal strength and the geological setting. A blend of statistical methods based on empirical experience and numerical modelling supported by field monitoring experience has been found to be useful for the design of pillars in this range.

Statistical methods based on empirical experience should be used with caution for pillars in the range four to six. Back analysis of the pillar behaviours observed in the Southern Lake Macquarie area (Mills and Edwards 1997) indicated that pillars in low strength roof and floor conditions are not as strong as similar sized pillars in strong roof and floor conditions. The warning of Galvin et al (1999) is particularly relevant for pillars in this size range.

Considerations of coal strength behaviour discussed in this paper indicate that:

- Coal can be characterised as having a cohesive strength component and a frictional strength component, each independent of the other.
- Pillar design methodologies should be applied with recognition of the characteristics of these two components, the external factors that affect them and their influence, in combination, on pillar behaviour.
- Small pillars that rely for their strength on the cohesive component of coal strength can reasonably be characterised using statistical methods, factors of safety and probability of failure.
- The factor of safety chosen should recognise the confidence with which the pillar loading and strength characteristics of the coal pillars are known rather than reliance solely on a generalised probability of failure criteria. If the loading can only be estimated approximately and the coal pillar strength is not known, the factor of safety chosen should be much higher than if the loading and strength are well constrained by field measurement.
- Large pillars that rely for their strength on generating confinement to their core should be designed with consideration for the geological setting in which they are located. A factor of safety is not meaningful for a large pillar in strong roof and floor conditions because pillar strength continues to develop as the pillar becomes more heavily loaded.
• The pillar loading of interest for design relates to the serviceability of adjacent roadways even if the pillar itself has not technically failed. Numerical modelling provides a way to characterise this behaviour.

• Understanding of the geological setting, the ability of pillars develop confinement in this geological setting and strategies to limit panel convergence are much more relevant to the design of large pillars than factors of safety or probabilities of failure.

REFERENCES


ROADWAYS DEVELOPMENT AND MONITORING WITHIN DEEP COAL MINES IN THE CZECH REPUBLIC

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ABSTRACT: Use of the system of dual-use of gateroads represents a significant change in the hitherto preferred method of addressing of mining works development. The entire system is based on the principle of reuse of one gateroad during the process of mining of the pair of neighbouring mining panels. The basic condition for its possible application is the stability of the supporting reinforcement of the gateroad so as for it to be able to withstand the additional loads originating in conducting longwalls or other expected supplementary loads to ensure the stability of the mine working throughout its required service life. Sufficient stability of these workings is ensured by the high anchoring method. It is a supporting system of long, large-area, possibly atypical mine workings, in which the elements of supporting reinforcement are embedded in solid layers of higher overburden of the supported mine working. The following paper presents the hitherto results of the process of verification of suitability of deployment of the high anchoring method for the aforementioned purpose, which was carried out in situ in the gateroad 063 5348 of Paskov Mine.

INTRODUCTION

Exploitation of black coal at ever greater depths means a shift in both the geological and geomechanical conditions, which places great demands on the design, realization and load-bearing capacity of the supported mine workings. Another factor which leads to the need for an increase of the bearing capacity of the reinforcement supports is that gateroads are often used twice. If a mine working is to fulfil its purpose for longer, the adverse effects of stress and strain deformation effects on the rock mass can be limited by means of rod bolts and flexible anchors, or by means of an increase of reinforcement support load bearing capacity through stranded anchors in combination with steel arch supports.

The essence of high anchoring technology consists in the use of materials with the necessary strength characteristics, capable of transmitting high loads in the body of the anchors, both in the axial and radial direction, with concurrent sufficiently solid and resistant link between the anchor, the resin and the rock. High anchoring performs correctly only in well-driven mine workings, with properly secured over-laminations behind the steel arch supports. High anchoring consists of a steel arch plane with a reinforced hole, stranded anchor and a sealant, by means of which the anchor is clamped in the rock mass.

BASIC CHARACTERISTICS OF STRANDED ANCHORS

Anchors applied in the rock mass govern the behaviour of rocks and their layers based on the principles of reinforcement. Installing stranded anchors bonded in the bore allows for better reinforcement of rock structures and avoids their possible stratification, which significantly influences the stability of immediate overburden and bedrock of underground and mine workings (elimination of the rate of convergence) (Souček et al., 2012, Kukutsch et al., 2013).

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The extent of reinforcement of the rock layers may exceed due to the length of the used anchors several times the height of underground or mine working, roadway, and longwall face end.

The most common use in practice are for:

- reinforcing mine roadways and longwall face ends;
- reinforcement of outsized underground or mine workings - crosses, forks, branches and chambers, etc.
- reinforcement of underground and mine workings with long service life,
- reinforcement of long mine workings for dual-use for adjacent longwall mining,
- reinforced securing of roadways or mine workings with an increased tension, eg roadways in the face-gate contact, in front of the advancing longwall.

**PRINCIPLE OF PLANNING AND DESIGNING OF GATEROADS FOR DUAL-USE**

Dual-use gateroads represents a significant change in the existing conventional solutions to development and mining works not only within one seam, but generally the whole block. This implies the need to address the selection of gateroads for dual-use at the time of preparation of the plan of opening, development and extraction of a coal seam or block. Compliance with the temporal requirements for development and extraction using the dual-use gateroad method is of quite an extraordinary importance.

The driveage of gateroads designed for dual-use must be based on a technological project according to these main principles (Technical Standard No. 1, 2009):

- When selecting gateroads suitable for dual-use it is necessary to take into account geological and mining conditions in which the gateroad will be driven. The evaluation must include assessment of the climatic conditions in the given area and a detailed evaluation of the degree of risk of spontaneous combustion of the seam in question.

- Gateroads, which are intended for dual-use, must always be at the time driveage be reinforced by combined supports.

When designing gateroads for dual-use it is necessary to consider the fact that after crossing of the coal face of the first longwall it is necessary to immediately proceed to the construction of a protected rib along the gateroad. After coal extraction of the seam the rib becomes the essential protective element to ensure the stability, shape and profile of the mine working. Protective rib preferably best performs its function if it is built as close as possible to the rib of the working and creates a solid and stable unit.

The protective rib is usually of a width, using stacks of hardwood or floating of fast-curing materials, equaling seven tenths of the mined seam thickness. If the extracted seam thickness is less than 1.5 m, then the minimum width of a rib is 1 m. The number of steel arch settings and the density of props building in the area of rib is specified by the technological procedure for the commencement or starting up of the first longwall.

**GUIDELINES FOR THE SELECTION OF LONGWALLS AND GATEROADS INTENDED FOR DUAL-USE**

Selection of gateroads for dual-use is based on the layout of longwall panels in the given seam and / or its part (block, field). Gateroads for dual-use are those that are separated by neighbouring longwall panels. The gate of the previously extracted panel is the mining gate and the gate of the subsequently mined panel is the return gate. In the case of more consecutively mined adjacent longwall panels the gateroads designed for dual-use can create a whole system
in the seam / part of the seam. It is necessary for extraction of adjacent blocks, between which there is a gateroad intended for dual-use to be mined as much consecutively as possible. Large time lag between the extraction of the neighbouring blocks eliminates the positive effect of the extraction technology with dual-use of gateroads, since the relevant gateroad designed for dual-use is exposed for too long to the effects of rock mass pressures.

Dual-use corridors can be realized only in suitable geological conditions, which are given by thickness and gradient of the coal seam, the quality of the overlying and underlying rocks, and geomechanical properties of the rock mass. Under suitable conditions it is considered (Technical Standard No.4, 2011):

- seam thickness of up to 2.5 m, in a very favourable geological and geomechanical conditions up to 3 m,
- seam incline the minimum possible, thus the seam flat deposition (up to 22°), greater incline means worse conditions for the application of this technology,
- seam overlay preferably solid, not prone to multiple break-outs, suitable for application in the rockbolt supporting system in the gateroad in question,
- seam underlay formed by solid rocks so as to avoid floor lifting in the gateroad in question,
- low tectonic fracturing of the relevant part of the rock mass, in order to be able to ensure the stability of the relevant gateroads with available resources, in the needed sufficiently large cross-sections (up to about 30 m²),
- strain fields such that the gateroads intended for dual-use are not subjected during their necessary life by extremely high primary especially and especially induced strains from mine workings situated in the overlay, the seam itself in the underlay (residual pillars of any kind, edges of back ends and others),
- hydrogeological conditions - without or with little water content in the rock mass in the seam overlay,
- risk of anomalous phenomena in any case means a complication, however, does not exclude the use of dual-use gateroads.

Due to the limitations of this technology on the top area of the deposited seam it is necessary to consider in particular the risk of rock bursts and the risk of outbursts.

**MONITORING OF THE GATEROAD 063 5348**

Gateroad number 063 5348 was located in the seam 063 (17b) at Paskov Mine, which is a mine without the danger of rock bursts. This seam is included in the second coal and gas outburst risk class. The seam incline is variable from 18 to 25 degrees. The underlay of the seam consists predominantly of siltstone, and the overlay in the section of 830 to 690 m contained siltstone with fine sand and thin sandstone lamination, 690-490 m transitions into sandstone, 490-390 m siltstone with sandstone lamination and the section 390-330 m contained siltstone.

Mining of the longwall No. 063 607 (Fig. 1) was launched at the starting stationing of 825 m in the gateroad No. 063 5348. Average mined thickness of the longwall was 214 cm, the length of the face was 169 m and the directional length of the longwall was approximately 480 m. Average daily advance of the face was around three meters.
Mechanical extensometers

Installation of 10 units of checking extensometers, type TTW07S (Fig. 2) with a length of 12 m was performed in boreholes with a length of 12.5 m and a diameter of 35 mm in the gateroad No. 063 5348 with a spacing of 50 m in the stationing 837 m, 784 m, 737 m, 687 m, 637 m, 583 m, 537 m, 487 m, 433 m a 383 m (labelled E1 to E10).

The TTW07S extensometer is a three-level design type. The height of the anchor "A" is 4000 mm, the height of the anchor "B" is 8000 mm and the height of the anchor "C" is 12000 mm above the roof of the mine working.

Monitoring of the extensometers consisted in control of the color scale or its part located under the roof outside the reference scale of the extensometer and accurate reading on the scales "A" and "B", "C" in millimetres was performed by the employees in weekly intervals throughout the entire period of 063 607 longwall mining.
The records of measurements on scale "A" and "B" and "C" of mechanical extensometers clearly imply that the most extensive displacement of the overlying rock occurred in the course of transition of the longwall in the section of about 100 m from the initial breakthrough when in the case of the scale "A" the value of displacement ranged up to 75 mm (full retraction of scale "A"). The most noticeable displacement was recorded in extensometers No. 1, 2 and 3. The remaining extensometers registered displacement in the range from 5 to 25 mm.

Almost a similar nature and trend of the development of the measured results of displacement of the overlying rock can be seen also in "B" scales. The highest increases of the displacement values were recorded in the extensometers No. 1, 2, 3, which were showing values increased up to 75 mm (full retraction of scale "B"). The value of displacement of the overlying rock in the remaining extensometer with "B" scales did not exceed 40 mm.

The measured results imply that the overall value of displacement (A+B+C) exceeded the value of 225 mm in two extensometers. The overall value of displacement in the remaining extensometers is not so significant, however the damage of several extensometers complicates the interpretation of the rock mass behaviour.

We expect that the increases at the extensometers E1, E2 and E3 are caused by the influence of the horizontal stresses behind the advancing face, which manifest from the stationing of 720 m in a significant deformation of the mine working.

**Dynamometers**

Three measuring stations were established in the gateroad no. 063 5348, where two dynamometers were installed to monitor the load of the stranded anchors in mutual distance of 2 m.

Dynamometer by Sisgeo, L2M0 series, model designation 0L2M0705000 allows manual readings of values up to 500 kN. Dynamometers were installed in the central part of the roof arch of the steel arch support (Fig. 3).

Location of stations each with a pair of dynamometers was at the time prior to mining of the longwall No. 063 607 at stationing 550 m (marking of dynamometers D6, D5), 630 m (D4, D3), 690 m (D2, D1). During the transition of the face through the measuring station in the 690 m stationing the dynamometers D1 and D2 were damaged. Based on this fact an additional dynamometer was installed in the 424 m stationing (D7).
Dynamometer readout was performed in weekly intervals. With regard to the increase in values just before the approaching face, it was decided to readout values in each shift, i.e., ideally 4 times a day.

During longwall mining, the increase of values was observed only in the immediate foreground of the longwall (located within the influence of additional stress from the edge of the longwall face). However, in the field behind the longwall the pressure effects of the overlay resulted in the change of the values on the installed dynamometers, which show an increasing trend with the increasing distance of the longwall. The maximum value of additional load was 220 kN (i.e. 22 tons), minimum value was 0 kN. The increase of values in dynamometers D3 and D4 depending on the advance of the coalface during two months is shown in figure 4.

![Figure 4: Development of load of dynamometers D3, D4](image)

**Vertical and horizontal convergence**

In places of the installed dynamometers were fitted with points for measuring vertical and horizontal convergence of the mine working. The measuring points for vertical and horizontal convergence in the gateroad No. 063 5348 were located in the monitoring places on hydraulic dynamometers, i.e. in the stationing 550 m, 630 m, 690 m.

Measurement of vertical and horizontal convergence was performed at weekly intervals.

In terms of stability of the gateroad No. 063 5348 the development of the measured values of vertical and horizontal convergence before the longwall was of other nature than in the places where convergence measurement does not take place, but the deformation of the mine working profile was very pronounced. While the convergence and slippages of reinforcement steel arches at the connecting stirrups of the monitored gateroad ahead of the face have been showing nearly identical results (reduced heights and widths: 150 mm, 80 mm, 15 mm slippages), maximum values of convergence on the unmeasured sites (area of extensometers 1-3, where there has been a maximum displacement) several times exceeding the values from the convergence profiles. In the section behind the face (stations 720-825 m) the devices
registered a decrease of the width and height of the profile of the mine working by 100 cm, respectively up to 200 cm (stationing 825 m, height 2.8 m, width 3.4 m - original profile height 3.8 m, width 5.36 m). The areas ahead of the longwall face and behind it were also subject of continuous floor brushing.

CONCLUSION

The article evaluates in a limited extent the partial results of monitoring and convergence of the gateroad No. 063 5348 of Paskov Mine. The hitherto results show that the issue of gateroads for dual-use in combination with high anchoring needs to be addressed, because it was confirmed that the high anchoring performs its function well only in a well driven mine working without overbreaks. The graph in figure 4 shows that the increase in load of the monitored anchor occurs when the monitored anchors get behind the edge of the longwall face. It is apparent from the observation of the mine working that the floor along with the steel arch supports are lifted before the advancing face, which results in a release of the installed stranded anchors. Judging according to the provisional results of the first approximation, we can state that the positive effect on the function of high anchoring and combined reinforcing systems could be achieved by highly pre-tensioned stranded anchors (at least to a value of 150 kN), which were a part of another experiment at Mine Paskov in the seam 080, in the gateroad No. 080 5253.

The most significant displacement of the overburden and the highest convergence of the profile of the gateroad No. 063 5348 were recorded in the area of extensometers E1, E2 and E3, which were caused by the influence of horizontal stress behind the advancing face, and which manifest from stationing of 720 m in a significant deformation of the mine working, and adverse geological conditions in the area (incidence of tectonics). The influence of geological conditions will continue to be the subject of further investigation based on the development of the monitored parameters. Objective evaluation can be made only by comparing the RQD parameter from the boreholes drilled ahead of the advancing face and boreholes performed in the same stations after passing of the longwall face.

The monitoring results presented in this paper demonstrate that the pressure conditions in the gateroad no. 063 5348 reflect both the deformation of the mine working behind the face, as well as the increase of the values on the installed extensometers and dynamometers, but only in the immediate vicinity of the face. We have to admit that pitfalls of the monitoring carried out consist in the fact that it is not always possible to combine operation and observation, so the extensometers are subject to damage (release, tearing of indicators), rebuilding of SA reinforcement with the identified convergence marks etc.

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REFERENCES

ASSESSMENT OF DEVELOPMENT ROADWAY ROOF CONDITIONS AT AN OPERATING UNDERGROUND COAL MINE USING NEURAL NETWORK ANALYSIS

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ABSTRACT: Considering that the majority of Australian longwall mines are currently roadway development constrained, understanding of the key geotechnical parameters that determine the roadway roof behaviour is often critical to the success of modern longwall operations. Crucial to improving this understanding is geotechnical characterisation. This process typically evolves with time and experiences at any underground coal mine and is necessary to understand the variation of the rock mass across a mining area. Based on local experience it is possible to improve forecasting of how similar geotechnical areas will behave and subsequently, the types and densities of support required to maintain roadway serviceability. There are numerous methodologies to characterise a rock mass and determine geotechnical domains using site-based data, which can vary from a simplistic single variable back analysis to more complex multivariate approaches. In a relatively isotropic and benign roof environment, simplistic models have been proven to be effective to provide an acceptable understanding of the change and variability in the rock mass and associated roof behaviour. However, with more challenging, weaker rock masses, where there are multiple independent features driving roof behaviour, the more complex statistical based back analysis approaches are more appropriate in order to accurately define different geotechnical domains. In this case study at Grosvenor Mine, a novel application of complex multivariate statistics in the form of a neural network analysis is shown to provide a useful and significant improvement in forecasting of the as-mined roadway conditions. This example indicates that in complex and challenging geotechnical environments, the application of complex analyses to characterise and understand the ground conditions is a promising potential area of further research, particularly with the advances being made in artificial intelligence more broadly.

BACKGROUND

Grosvenor Coal Mine is located near the township of Moranbah in Queensland’s Bowen Basin, approximately 150km west of the coastal city of Mackay. Grosvenor (GRV) mine, as with three other longwall mines in the vicinity, extracts the Goonyella Middle Seam (GMS) and has a mine life in excess of 25 years. The GMS ranges in thickness from 4.2 to 5.6 m thick across the GRV lease which is at the lower end of the thickness spectrum of existing GMS mines. Conventional underground development commenced in 2014 following completion of the mine access drifts with a tunnel boring machine. The ground conditions experienced at GRV have been highly variable, with certain areas proving challenging to develop due to weak rock masses and stress driven roof deformation, which is not typical of existing GMS longwall mines. The deformation usually occurs soon after development, biased to the central portion of the roadway and is typically in the form of a buckling immediate roof with associated centreline cracking, roof bagging/sagging, and elevated levels of roof extensometer Tell-Tale (TT) displacements. This deformation is typically experienced in the Cut Throughs (C/T), due to being adversely orientated to the major horizontal stress. In contrast, the headings are orientated almost parallel

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to the major horizontal stress, with improved conditions experienced as a result. Examples of
typical C/T roof deformation and Tell-Tale (TT) displacement are illustrated in Figure 1 and
Figure 2.

Prior to commencement of mining at GRV, initial geotechnical characterisation was based on
the geological distribution of coal plies and seam splits, as the adjacent Moranbah North mine
had encountered significant ground control difficulties in the vicinity of an upper seam split. This
seam split was also present at GRV across a broad area referred to as the seam split zone. The GMS at Grosvenor is made up of five main plies, moving sequentially from ply-1 to ply-5
starting at the top of the GMS. In the seam split zone, ply-1 gradationally splits away from the remainder of the GMS, up to a maximum of 5m above the ply-2 with carbonaceous stone interburden between the plies. As such, where ply-1 is logged more than 200mm away from the GMS, it is classified as "Domain A". Domain A trends across the centre of the lease in a broad NWN/ENE direction, with the areas outbye and inbye classified as “Domain B Shallow” and “Domain B Deep” respectively (see Figure 3) implying a similar geotechnical environment either side of Domain A.

![Figure 3: Grosvenor geotechnical domains](image)

After several kilometres of development had occurred in all three geotechnical domains, it was observed that the original boundaries did not correlate as strongly as expected with the ground conditions experienced. As mining progressed and more experiences were gained, several key variables were identified that determine the development roof behaviour other than the distribution of coal plies. As such, a project was instigated to review the available data with a view to incorporating these factors into defining the geotechnical domain boundaries. Although several factors had been identified, the variables were independent of each other, and the combined influence of each on excavation behaviour was a complex interaction. In an attempt to cater for this complexity, a statistically based Neural Network (NN) analysis was tested. Within a NN, algorithms are used via machine learning to define a relationship between the identified variables and the outcome (development roadway roof behaviour).

NN’s are currently widely used across a range of different industries to establish patterns and relationships in datasets as a form of artificial intelligence with the use currently expanding at an exponential rate in conjunction with the expansion in collation and storage of data.

Existing examples include international airlines analysing past food consumption and using NN to predict food usage on long-haul flights thus allowing for optimisation of food inventory and minimising fuel burn. Another everyday example is banks detecting fraudulent transactions based on past purchasing habits. Although growing rapidly in other industries, and no doubt being developed in house at many large organisations, there are currently only a few published examples of NN’s utilised in mining applications. A relevant example was developed by Rankine (2004), in which a NN was produced for the prediction and optimisation of cement backfill at BHP’s Cannington Mine in north-west Queensland. Broadly, the benefit of a statistical machine learning based approach, such as NN, to understanding datasets is that instead of computers being given explicit instructions on how to analyse data, i.e. closed form, they are given a set of rules. This allows self-training and the ability to iteratively learn from datasets and gain insight
that would otherwise remain hidden. The potential for finding additional insight is ongoing as further data is collected over time and incorporated into the NN (ADL, 2018).

MODEL PARAMETERS

Overview

The premise for parameter selection is to be as least subjective as possible to drive repeatability and consistency into the project. As the final model is aimed at forecasting difficult roof conditions in development, three potential measures of outcome were explored. The first was a qualitative estimation of the roof conditions, i.e. good, moderate or poor. However, as these measures are highly subjective, it was disregarded. A rating system was also considered, based on several observable features such as excessive extensometer movement, centreline cracking, both presence and magnitude, TARP level, etc. However during the project it was also identified that the identification of these features still introduced unsuitable levels of subjectivity into the data gathering process.

The third option, and that selected for use as the sole dependent variable (outcome in the NN), is total roof extensometer displacement. It is acknowledged that there are certain limitations when using roof extensometer data as an indicator for roof behaviour, including; installation quality with respect to monitoring height and anchorage, anchorage slippage, absence of lateral displacement (shear), variance in instrument placement (location in roadway in relation to where deformation is occurring) and lack of insight into the failure mechanisms due to a single data point. However, as GRV has a very high density of monitoring devices (approximately 25 m spacing), and subsequently a large number of data points, it was hypothesised that meaningful relationships can be determined, and obvious outliers identified and addressed, despite these limitations.

Based on the experience at Grosvenor and Moranbah North Mines, the following six independent variables have been identified as influencing roof deformation (convergence):

- Roof unit thickness
- Primary roof bolt length
- Long tendon type
- Long tendon spacing
- Roof Unit 1 composition
- Roadway stress environment.

All the above parameters are readily available from existing downhole geophysical logs, underground mapping, rock mass characterisation, engineering calculations and geotechnical design. As such the model can be developed, implemented and updated without the need for a significant amount of additional work to be performed or specialist skills.

Roof unit thickness

Due to the range in seam thickness and nominal development cut height (3.6 m), the immediate overlying strata above the working section at GRV comprises of coal (plies 1 and 2 of the GMS) followed by a transitional sequence of carbonaceous mudstone and siltstone into bedded siltstones and sandstones. The thickness and composition of the immediate roof changes significantly across the lease in geological terms; however, by utilising physical measurements from borehole geophysical logs, three discrete geotechnical units (ROF1, ROF 2 and ROF3) can be readily determined. Using physical measurements, rather than geological logs which may vary between geologists based on logging, much greater consistency is achieved.
Although in reality the excavation height will range between 3.6 m to 4 m above seam floor, for standardisation, the cut profile has been selected as 3.9 m above the GMS floor for determining the base of ROF1 (cut roof horizon). This assumes 0.3m of coal left in the floor which is the targeted development horizon at GRV.

ROF1 refers to the “clean” coal overlying the cut profile and is identified where the density log is relatively low and consistent (<1.5 g/cm³). The thickness of ROF1 displays a typical pattern of variation along the gateroads, initially approximately 0.6 m thick (Domain B Shallow), thinning to less than 0.1 m in areas in the centre of the gateroads (Domain A), before thickening to >1m at the inbye end of the panels (Domain B Deep).

Above ROF1 lies ROF2, a mainly carbonaceous siltstone which is transitional in nature and characterised by weak contacts between units. This unit is typically highly bedded and laminated, and forms a weak rock mass, evidenced by both low material strength (uniaxial compressive strength or UCS) and diametral point load strength. This also varies significantly in thickness along the gateroads, showing a typically inverse relationship to ROF1. Generally, where ROF2 is at its thickest (> 3m), ROF1 is at its thinnest. It is within this zone (Domain A) that the most challenging roof conditions are encountered in development. As the thickness of both ROF1 and ROF2 appear to correlate well with experienced conditions, they have been included as independent variables for the model. An example of the picked ROF units can be seen in Figure 4 from borehole DDG190R.

Figure 4: Picked roof units

The variation in these ROF units across the GRV lease is readily determined from the geophysical logs. Figure 5 through Figure 7 show a representative borehole log (density, sonic derived UCS and gamma) from each of the geotechnical domains at GRV. Note the variability in ROF thickness for each domain. While the evolution of the ROF units has continued over
time, it is evident that other variations of note occur in each domain, essentially creating sub domains, however these have not yet been incorporated into the NN.

Figure 5: Geophysical logs (Domain B Shallow)

Figure 6: Geophysical logs (Domain A)
One variation worthy of note is the change in gamma response for ROF 1 between Domain B Deep and Domain B Shallow, with Domain B Deep having a much lower response. This is mainly driven by the banded nature of the coal in Domain B Shallow and “clean” coal in Domain B Deep, indicating that the domains are not similar, in both lithology and behaviour. A thick clean coal beam is associated with increased roof stability at all GMS mines and GRV is no exception to this characteristic. The gamma content of the coal is discussed further below. Figure 8 and Figure 9 show the thickness distribution of ROF1 and ROF2 respectively across the lease.
Figure 9: ROF2 thickness (m)

Primary roof bolt length

Various lengths of roof bolt have been utilised at Grosvenor. The initial bolt length was selected as 1.8 m, however 2.4 m bolts have been utilised on install roads and in some particularly poor areas, and recently there has been a wholistic change to 2.1 m roof bolts. In general, it is recognised within the industry that the longer bolts will provide improved roof stability and behaviour, with the 2.1 m bolt length most common. However, individual sites must assess the benefits of a longer bolt with the disadvantages such as costs, equipment limitations (clearance, ergonomics) and the need for additional resin cartridge lengths to encapsulate a longer bolt.

The benefit of a longer bolt can be explained mechanistically with the following equation after Canbulat and Van Der Merwe, 2009.

\[
\tau_{\text{MAX}} = \frac{1}{2} \rho g \left( h + h_1 \right) L \tag{1}
\]

Where,
- \( \tau_{\text{MAX}} \) is the maximum shear stress within the bolted beam
- \( \rho \) is the density of material (kg/m\(^3\))
- \( g \) is gravitational acceleration (m/s\(^2\))
- \( h \) is the built beam thickness (m)
- \( h_1 \) is the height of softening (m)
- \( L \) is the beam width (m)

Increasing the bolt length will increase the built beam thickness (h), as such reducing the maximum shear stress within the bolted horizon. This concept is shown visually in Figure 10, which has been generated using a consistent height of softening of 5 m and a 5.4 m roadway width. Based on this assessment, changing from a 1.8 m bolt to a 2.4 m bolt can result in a reduction in maximum shear stress by 18%. 

University of Wollongong, February 2019
Long tendons

Two distinct types of cable bolts have been utilised at Grosvenor, due to varying strategies of strata control. The first type was an 8m end anchored cable bolt, with the top 2 m resin anchored and the bottom 2.5 m near the collar post grouted, leaving a 3.5 m free length in the cable. Due to the free length in the bolt, it behaves as a softer support unit accommodating greater levels of roof movement for the same applied load. The other type of cable that is now routinely used is an 8m full column grouted cable bolt. This type of cable is the most commonly used for roadway development in Australia. This is an active, stiffer support unit due to the fact that the cable is tensioned with resin on installation and the remaining length of the cable is encapsulated with cementitious grout post drivage.

As each cable is designed to control the roof by different methods, one aimed at accepting a higher level of roof movement, and the other trying to prevent movement, the installed cable type will have a significant impact on both the allowable tell-tale displacement and the magnitude experienced.

Long tendon density

Fundamentally the density of ground support installed based on spacing, length and capacity (ultimate tensile strength) has been shown empirically to have a positive correlation with roadway stability irrespective of the type of support utilised (Frith and Colwell, 2009). Based on the findings of Frith and Colwell (2009), a higher density of primary cables should provide a higher level of reinforcement, leading to lower levels of roof deterioration.

Roof unit 1 composition

As discussed previously, the current domain nomenclature implies that the areas inbye and outbye of Domain A are the same apart from cover depth. However, development experiences have proven that this is not the appropriate interpretation. Domain B Shallow which is present in the first 10 to 15 pillars of the first five panels is located at relatively lower depth of cover (180-250 m) and typically behaves favourably on development in the headings. However,
isolated cut-throughs have converged significantly (>50 mm). In contrast, Domain B Deep conditions are considerably improved in both headings and cut-throughs, despite the depth of cover being twice that of Domain B Shallow (350-420 m). As mentioned in the preceding section, although both domains have thick coal roof, upon further investigation, a significant difference in the composition of ROF1 across the lease is evident, with the Domain B Shallow ROF1 exhibiting significant clay banding that is not present in Domain B Deep. This banding and stone infill are variable and are likely to contribute to the variable behaviour of roadways in Domain B Shallow, i.e. increased delamination and roadway convergence. To quantify this difference, the median gamma value for ROF1 is determined. The gamma log is sensitive to the higher levels of radiation from thorium adsorbed by the clay minerals and potassium content. As the banding in the coal at Grosvenor is typically clay based, the gamma log provides a reasonable indication of how strong and persistent the clay banding is (Kansas Geological Survey, 2017). The median gamma content for ROF1 is shown in Figure 11.

![Figure 11: ROF1 median Gamma (API)](image)

Roadway stress

Intuitively, a weak roof environment at depth, such as that at GRV (low strength/stress ratio), is highly sensitive to changes in either parameter. To minimise the portion of roadway exposed to elevated levels of in situ horizontal stress, the gateroads at GRV are aligned approximately parallel to the major horizontal stress (033°), as such the cut-throughs are subject to higher stress concentrations in the roof and floor on development than the headings. This assumption appears to have worked in practice as even in the weakest roof areas, the headings experience far lower amounts of roadway deformation than cut-throughs, with total displacements >10 mm rare compared with >50 mm common in the adjacent cut-throughs. As the magnitude of the horizontal stress acting across the roadway has shown to have a dramatic influence on roof behaviour, it has been included as an input for the model.

The following equation has been used to determine horizontal stress acting in each roof unit, after Nemcik et al, 2005:

$$\sigma_H = \frac{\nu}{1-\nu} \times \sigma_V + E \times TSF_H$$  \hspace{1cm} (2)
where:

- \( \sigma_H \) = Major horizontal stress (MPa)
- \( v \) = Poisson’s ratio
- \( \sigma_V \) = Vertical stress (MPa)
- \( E \) = Young’s Modulus (GPa)
- TSF\(_H\) = Tectonic stress factor for major horizontal stress component

The values for \( v \) and \( E \) for ROF 1 and 2 have been determined based on averaging site wide laboratory data. There is some level of variability for these values across the lease, however for simplicity the mean of each ROF unit is used. After determining the mean stress acting in each unit, a weighted average in the bolted interval was calculated to determine the ratio of the major and minor horizontal stress to vertical stress. Once these ratios were calculated, it was possible to determine the total magnitude of the horizontal stress acting across each roadway orientation using the following equation, as summarised by Hoek, 1980:

\[
\sigma_R = \frac{\sigma_H + \sigma_h}{2} - \frac{\sigma_H - \sigma_h}{2} \times \cos(2\beta)
\]

where:

- \( \sigma_R \) is the resultant horizontal stress
- \( \beta \) is the difference in orientation between the roadway and the major horizontal stress

After calculating the horizontal stress for each roadway analysed, a graph can be produced showing resultant stress against depth of cover, to gain useful insight into the distribution of stresses in both headings and cut-throughs, for the expected variation in the ROF unit thickness. This can be seen in Figure 12.

![Figure 12: Resultant roadway stress along gateroads](image)

The graph above shows that there is a gradual increase of the stress with depth of cover, however the trend begins to reverse at the \( \sim \)350 m mark despite cover depth continuing to increase inbye. This observation coincides with a thickening of the less stiff ROF1 unit from Domain A to Domain B Deep, with the lower modulus coal attracting less stress, resulting in a lower resultant stress value. This lower level of stress applied to ROF1 is suspected of contributing to the improved conditions in the inbye areas of the gateroads (Domain B Deep).

RESULTS
Model fit

A total of 139 cases were used to develop the model using the NN approach. This consisted of 111 training cases and 28 cases to test the model on. The utilised software provides inbuilt analytical tools to identify the quality of the model generated in order to determine how accurately the dataset can be represented by a given model.

For the testing cases, mean absolute error was 4.2 mm with a standard deviation of 5.2 mm. This is a relatively good fit, indicating that on average the model can predict the values from the testing database within 4 mm, and that a meaningful relationship has been developed between the indicator variables and the dependent variable (total tell-tale displacement). This relationship is illustrated in Figure 13.

![Figure 13: Predicted vs Actual TT movement (training cases)](image)

The red line represents a perfect case of actual tell-tale movement equalling predicted, and the blue line the trend of the data set. The correlations are very similar, although the predicted tell-tale movements tend to be lower than the actual for elevated levels of displacement (>50 mm). This may be due to the limited data points at higher overall displacements and suggests that the model may somewhat underestimate at higher displacement levels. It is noted that as the roof displacements gained from this model are currently used to identify different geotechnical domains, it is assessed that minor variations will be managed in the development TARP.

Although this analysis is useful for determining model fit, it does not suggest how accurate it is against points outside the training database. As such, the software also runs an analysis on the predicted vs actual for the testing cases, which were excluded from training the data set. This can be seen in Figure 14.
This graph shows a similar trend as Figure 13, with a reasonably close relationship at the lower tell-tale movements, however, the predicted displacement again tends to be lower than the actual at the higher levels. The residuals of this data set can also be plotted, which shows quantitatively the errors in the model based on actual data, as displayed in Figure 15. The residual refers to the difference between the actual and predicted values. The higher the residual, the further the predicted value is from the actual, and provides an indication of the model's precision.

The above figure shows that in 75% of the cases the residuals are less than plus/minus 10mm of the actual measurement. At the higher magnitude displacement levels, i.e., in the remaining 25% of the cases, the residuals lie between plus/minus 20mm of the actual. Based on the preceding graphs and analysis, it is reasonable to conclude that the model is suitable for the purpose of identifying different domains that are likely to be subject to higher levels of deformation in comparison to other areas.
Modelled Tell-tale Data

Long sections for two gateroads have been generated, comparing the maximum cut-through tell-tale displacement to the modelled tell-tale data.

![Figure 16: Maingate 102 long section](image1)

![Figure 17: Maingate 103 long section](image2)

It is evident from these long sections that there can be significant localised variation between adjacent cut-throughs in terms of total displacement that the model cannot replicate. The cut-throughs are typically 125m apart, and it is clear from the tell-tale data that the overlying lithology and/or other operational factors vary over this distance. Borehole spacing, which is what 4 of the 6 indicator variables are based on, are often spaced further than 125m. This is a limitation for capturing the localised variations occurring in the rock mass at this scale and subsequently limiting the precision of the model.
The Maingate (MG) 102 long section also confirms the observation in the previous section that the high levels of tell-tale movement do not appear to be repeatable in the model. Yet the model does show a distinct zone of elevated displacement consistent with the actual data within Domain A, indicating that different zones can be identified with the predicted values.

The overall trend for both panels of actual vs. modelled displacement are reasonably similar, indicating that the model could be used to identify zones at increased risk of high levels of roadway convergence prior to mining.

**Tell-tale displacement prediction in MG104**

The predicted tell-tale displacements for the entire length of MG104 can be seen in Figure 18, with the actual data plotted in blue up until ~2600m chainage (CH). Note that at the time of the model development, the face chainage was approximately 900m. The model indicated that the roof conditions were expected to be reasonably consistent until approximately CH1500m (close to 15 C/T), where a sudden increase in roof convergence was expected. It is considered that this increased convergence will be driven by a combined effect of the thickening of ROF2, thinning of ROF1, and increase in horizontal stress in the immediate roof due to the higher stone content. The elevated levels of displacement were expected to continue to approximately CH2400m (23 C/T), where the model predicts a significant reduction in roof deformation, largely based on a thickening of the ROF1 unit. Pragmatically this analysis would suggest that poorer conditions and more frequent TARP triggers would be expected also from 1500-2300CH, as well as likely poorer conditions in the gate end roadways during LW retreat.

It can be seen that the actual vs modelled tell-tale displacement varies rather significantly in magnitude in the identified weak zone. However, the underground observations closely matched the forecasted conditions as stated above, in that between 15 C/T and 22 C/T, significant deterioration was observed, with frequent TARP responses and additional support required to maintain roadway serviceability. Again, this confirms the previously stated findings that the model cannot replicate exact tell-tale displacement magnitude or variability, however it can correctly forecast the poor zones typical to Domain A along the gateroad length.

**Figure 18: MG104 long section (actual vs modelled TT displacement for C/Ts)**

**CONCLUSIONS**
Based on the analysis detailed above, this project has successfully defined a relationship between a set of independent variables, and the total recorded displacement of a roadway under development loading conditions with the total displacement used as a proxy for ground conditions. This model is based on actual data from an operating underground mine site and has been successfully used to improve the forecasting of tell-tale displacements in future roadways at this mine site. This allows for improved operational planning that otherwise would not be possible based on existing methods of geotechnical characterisation. As with any system that incorporates actual data, this NN model can and should be updated as additional information becomes available.

It is found that the model’s accuracy is acceptable. However, at the higher ends of the predicted tell-tale movements, the model somewhat underestimates the displacement. As such, the model should not be used to assess the exact displacements in a roadway under given conditions. It is recommended that this model should only be used as another tool to identify geotechnical zones within panels that are likely to encounter more difficult ground conditions in comparison to other areas. These zones can be used in lieu of the existing geotechnical domain boundaries. Improved ground characterisation when communicated to the workforce clearly prior to mining, will increase the likelihood of the appropriate support densities and TARP responses being implemented, while also assisting the mine planning process to predict cut rates that reflect realistic targets.

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REFERENCES

NUMERICAL SIMULATION OF STRESS DISTRIBUTION IN LONGWALL PANELS DURING THE FIRST CAVING INTERVAL

Sadjad Mohammadi¹, Mohammad Ataei², Reza Kakaie³, Ali Mirzaghorbanali⁴, Naj Aziz⁵ and Ashkan Rastegarmanesh⁶

ABSTRACT: Reliable prediction of induced stress distribution in longwall panels enhances safety in longwall mining. This paper presents the results of numerical simulation intended to examine stress distribution in terms of peak abutment pressures during the first caving of longwall mining. Longwall mining was simulated by incorporating Universal Distinct Element Code (UDEC). Several conceptual models were developed and subsequently analyzed to investigate the effects of five critical parameters on peak abutment pressures. Critical parameters that were studied as a part of this investigation included roof strata uniaxial compressive strength, immediate roof height, spacing of bedding planes and Vertical and horizontal in situ stresses. The results of numerical simulation increased the current understanding of rear and front abutment pressures in longwall mining under various geologic conditions.

INTRODUCTION

Study of the induced stress distributions in the vicinity of a longwall panel plays a fundamental role in the understanding of mining mechanics due to its direct impact on safety and productivity. The stress distribution in term of abutment pressures (Figure 1) has a direct effect on the failure mechanism, roof control, location and stability of the gateroads and frequency and intensity of dynamic accidents such as gas outburst and rockburst in working faces. Accordingly, the accurate prediction of the abutment pressures enhances safety in longwall mining.

There are direct (e.g. stress measurement, experimental simulation, simplified elastic–plastic or constitutional damage calculation) and indirect (e.g. tunnel deformation and support pressure measurement) methods to measure the abutment pressures (Gao et al., 2013). Up to now, several equations were presented in the literature to predict abutment pressures (Salamon, 1963; Peng and Chiang, 1984; Jeramic, 1985; Wilson, 1986; Mark, 1990; Heasley, 1998; Gil, 2013; Verma and Deb, 2013; Zhu et al., 2015). Furthermore, some other researchers have studied numerically the effect of various pertinent parameters on the abutment pressures (Singh and Singh, 2010; Gao et al., 2013; Ju et al., 2015; Ji et al., 2016; Jiang et al., 2018).

In the literature, no studies have been carried out to investigate the effects of critical parameters on the abutment pressures with regards to the existence of different strata in the immediate roof. It is noted that discontinuous methods are more appropriate for simulating the progressive caving of strata due to longwall mining. Accordingly, a systematic numerical study incorporating Universal Distinct Element Code (UDEC) was performed to investigate the effects of five critical parameters on the rear and front Peak Abutment Pressures (PAP) during the first caving interval considering the different composition of immediate roof strata. Critical parameters that

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were studied included the roof strata uniaxial compressive strength, immediate roof height, spacing of bedding planes and \textit{in situ} stresses.

**Figure 1: Abutment pressure state (modified from Yavuz, 2004)**

**SIMULATION REQUIREMENTS**

The two-dimensional Universal Discrete Element Code (UDEC) was used to simulate the first caving event due to longwall mining. For this purpose, an \textit{initial condition} was defined, then, to study each parameter, only the value of that particular parameter was changed and the other parameters were kept constant under the \textit{initial condition}. Table 1 illustrates the \textit{initial condition}. Geometry and boundary conditions of the constructed models are shown in Figure 2.

**Table 1: Basic conditions**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal seam thickness</td>
<td>2 m</td>
</tr>
<tr>
<td>Immediate roof height</td>
<td>5 m</td>
</tr>
<tr>
<td>Mining depth</td>
<td>300 m</td>
</tr>
<tr>
<td>In situ stresses ratio</td>
<td>1</td>
</tr>
<tr>
<td>Bedding planes spacing</td>
<td>1 m</td>
</tr>
<tr>
<td>Number of cross joint</td>
<td>1</td>
</tr>
<tr>
<td>Joint set dip</td>
<td>90°</td>
</tr>
<tr>
<td>Joint set orientation</td>
<td>Parallel to the face</td>
</tr>
<tr>
<td>Joint set spacing</td>
<td>1 m</td>
</tr>
<tr>
<td>Joint set persistence</td>
<td>0.5 m</td>
</tr>
</tbody>
</table>

**Figure 2: Geometry and boundary conditions of the numerical model**
In order to consider a variety of strata in the immediate roof, four types of immediate roofs were studied as shown in Figure 3. These types have been selected in such a way that immediate roof strata that was weak, or strong or that various components could be considered.

![Figure 3: Four types of immediate roof](image)

In the models, the rock blocks obey strain-softening (elastic-brittle-plastic) constitutive model with an ultimate and residual strength defined by modified Mohr-Coulomb strength criterion. Discontinuities follow Mohr-Coulomb constitutive law. The mechanical properties of the rock mass and discontinuities (Table 2 and Table 3) for this study were deduced from the mean of field data compiled for various panels and typical values extracted from the literature.

### Table 2: Mechanical properties of rock blocks

<table>
<thead>
<tr>
<th>Rock</th>
<th>$\sigma_i$ (MPa)</th>
<th>E (GPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$\nu$</th>
<th>C (MPa)</th>
<th>$\phi$ (°)</th>
<th>$\psi$ (°)</th>
<th>$C_r$ (MPa)</th>
<th>$\phi_r$ (°)</th>
<th>$\psi_r$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siltstone</td>
<td>10.00</td>
<td>3.10</td>
<td>2600</td>
<td>0.25</td>
<td>2.21</td>
<td>37.07</td>
<td>5</td>
<td>0.22</td>
<td>24.71</td>
<td>3.33</td>
</tr>
<tr>
<td>Shale</td>
<td>36.00</td>
<td>11.16</td>
<td>2600</td>
<td>0.25</td>
<td>8.311</td>
<td>38.91</td>
<td>5</td>
<td>0.83</td>
<td>25.94</td>
<td>3.33</td>
</tr>
<tr>
<td>Fine Sandstone</td>
<td>75.00</td>
<td>23.25</td>
<td>2600</td>
<td>0.25</td>
<td>14.30</td>
<td>45.73</td>
<td>5</td>
<td>1.43</td>
<td>30.50</td>
<td>3.33</td>
</tr>
<tr>
<td>Coarse Sandstone</td>
<td>150.00</td>
<td>46.50</td>
<td>2600</td>
<td>0.25</td>
<td>28.61</td>
<td>45.73</td>
<td>5</td>
<td>2.86</td>
<td>30.50</td>
<td>3.33</td>
</tr>
<tr>
<td>Floor and main roofs</td>
<td>120.00</td>
<td>37.20</td>
<td>2600</td>
<td>0.25</td>
<td>24.20</td>
<td>43.00</td>
<td>5</td>
<td>2.42</td>
<td>28.67</td>
<td>3.33</td>
</tr>
<tr>
<td>Coal</td>
<td>15.00</td>
<td>2.00</td>
<td>1500</td>
<td>0.4</td>
<td>5.80</td>
<td>15.25</td>
<td>2</td>
<td>0.58</td>
<td>10.20</td>
<td>1.33</td>
</tr>
</tbody>
</table>

$\sigma_i$: intact compressive strength; E: Young's modulus; $\rho$: density; $\nu$: Poisson's ratio; C: cohesion; $\phi$: angle of internal friction; $\psi$: angle of dilatancy; $C_r$: residual cohesion; $\phi_r$: angle of residual internal friction; $\psi_r$: angle of residual dilatancy

### Table 3: Mechanical properties of discontinuities

<table>
<thead>
<tr>
<th>Parameters</th>
<th>C (Mpa)</th>
<th>$\varphi$ (°)</th>
<th>$\sigma_t$ (MPa)</th>
<th>$K_n$ (GPa/m)</th>
<th>$K_s$ (GPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>0</td>
<td>30</td>
<td>0</td>
<td>20</td>
<td>4</td>
</tr>
</tbody>
</table>

C: cohesion; $\varphi$: angle of internal friction; $\sigma_t$: tensile strength; $K_n$: normal stiffness; $K_s$: shear stiffness

### RESULTS

**Effect of strata UCS**

The coal strata are grouped into several composite layers which have diverse thicknesses with different and complex mechanical and caving behaviour. Different strata properties have different influences on the immediate roof properties. Therefore, to consider the effect of such condition in overall strength of the immediate roof, the Equivalent Immediate Roof Strength (EIRS) was defined as the thickness-weighted average of the roof strata uniaxial compressive strength as follows:
\[
EIRS = \sum_{i=1}^{n} t_i \times \sigma_{ic} / \sum_{i=1}^{n} t_i
\]

where \( t_i \) is the thickness of the \( i \)th stratum (m), \( \sigma_{ic} \) is the Uniaxial Compressive Strength (UCS) of the \( i \)th stratum (MPa), and \( n \) is the number of stratum within the immediate roof.

Figure 4 shows the results of PAPs modelling for the studied roofs (in which its EIRS are 25.6, 56.2, 84.2, and 150 MPa, respectively) in the initial condition.

Effect of immediate roof height

From Figure 4 it is concluded that an increase in the overall strength of the immediate roof (EIRS) produces higher PAPs. The findings show that the front PAP increased from 15.15 MPa to 19.60 MPa with an increase in EIRS from 25.60 to 150 MPa. The rear PAP inhabits a similar trend, in which PAP varies from 14.74 MPa to 19.41 MPa. Moreover, it is observed that the rear PAP is always less than the front one.

Effect of immediate roof height

The immediate roof height (which is usually equal to the caving height) is correlated with extraction height as followS:

\[
h_{im} = \frac{m}{K - 1}
\]

where \( m \) is the extraction height (m) and \( K \) is the bulking factor of immediate roof.

Peng and Chiang (1984), based on field investigations, stated that the bulking factor of coal measure rocks varied between 1.1 and 1.5. Shabani mashcool (2012), based on the available literature, proved that the porosity of caved materials is approximately 0.3 which corresponds to a bulking factor of 1.43. Consequently, the immediate roof height would be roughly 2.5 times the extraction height which is also the assumed value in this study. Accordingly, in order to investigate the effect of the immediate roof on the PAPs, three heights (5, 7.5, and 12.5) were considered (Figure 5).

Contingent upon Figure 5, it is noted that there is a decreasing trend in the rear and front PAPs with an increase in immediate roof height, and supposedly, an exponential decay function could describe the decreasing trend of the PAPs with respect to immediate roof height for all of the studied roofs. Moreover, it is evident that the gradient of the trend line in strong roofs (roof 3 and 4) is higher than that of the weak roofs (roof 1 and 2).
Effect of bedding planes spacing

The bedding planes in the immediate roof were simulated by horizontal persistent joints with three mean spacing (0.25, 0.5, and 1 m). It is noted that the bedding planes spacing was presumed constant in the entire immediate roof. The bedding planes influence on the PAPs is illustrated in Figure 6.

Figure 5: Peak abutment pressures versus immediate roof height

a. rear PAP

b. front PAP
According to Figure 6, the rear and front PAPs observed during first caving event show an increasing trend with increases in the mean spacing of bedding planes and furthermore, the relationship between PAPs and bedding planes spacing is of a logarithmic growth. In addition, it is also clear that the growth rate of the strong roofs (roof 3 and 4) are higher than that of the weak roofs (roof 1 and 2).

**Effect of vertical in situ stress**

A parametric study to assess the effect of in situ stress on the PAPs was done by varying mining depth. Accordingly, a simulation was performed using four depths values of 150, 300, 600, and 1000 meters which corresponds to vertical in situ stresses of 2.6, 6.5, 14.3 and 24.7 MPa (Figure 7).

Plots in Figure 7 show that the rear and front PAPs increases almost linearly with an increase in strata depth and consequently vertical in situ stress. Additionally, the graphs show that both PAPs increases drastically with increase in strata depth up to a value of 600 meters, however, when the depth of working increases to 1000 m, the PAPs increases gradually as well.
Finally, the effect of the horizontal in situ stress on the PAPs was conducted via using a parametric study of different in situ stress ratios ($K$). For this purpose, strata depth of 300 meters was assumed and $K$ was changed from 0.5 to 3 which correspond to a band of 3.25 to 19.5 MPa of horizontal in situ stress change (Figure 8).

Figure 7: Peak abutment pressures versus vertical in situ stress

Figure 8: Peak abutment pressures versus horizontal in situ stress

Figure 8 illustrates there is no clear relationship between PAPs and the horizontal in situ stress. These results show that while the horizontal in situ stress increases, both rear and front PAPs
fluctuate capriciously. Furthermore, the magnitude of the rear and front PAPs in strong roofs are always higher than that of the weak roofs irrespective of the horizontal in situ stress value.

**DISCUSSION**

The prime motivation for this study was to investigate the relationship between the rear and front abutment pressures and some critical parameters. Different types of roofs in terms of strength and composition were taken into account while incorporating discontinuous by numerical modelling. Undoubtedly, the obtained values are not applicable to all cases, however, the general trends of the relationships are valid.

Figure 4 showed that there is a direct relationship between EIRS and PAPs. This relationship should be interpreted by taking the first caving interval into consideration. The effect of an increase in EIRS is an increase in the first caving interval. Consequently, based on the principals of potential energy balance, the PAPs are higher. Later, the indirect relationship between the immediate roof height and PAPs could be inferred from Figure 5. This result is in line with the analytical results of Majumder and Chakrabarty (1991) and numerical findings of Singh and Singh (2010). Moreover, Figure 6 indicated that the result of an increase in the bedding planes spacing would be higher PAPs which reflect the direct correlation between these two variables. This finding confirms the results of the numerical modelling of Gao et al. (2014). In addition, a direct relationship between the vertical in situ stress (which corresponds to strata depth) and PAPs is concluded from Figure 7 which is in accordance with the findings of Singh and Singh (2010). Meanwhile, the PAPs and main caving span are not always correlated directly and it is necessary to take into account the working depth. In a constant depth, an increase in the main caving span of a given roof is encountered with an increase in PAPs (similar to the interpretation of the results of the effect of strata UCS). Nevertheless, when the mining depth increases, the main caving span decreases, however, PAPs will be higher. Figure 8 showed there is not a clear relationship between the horizontal in situ stress and PAPs. This result is inconsistent with the numerical modelling results of Singh and Singh (2010). Gao et al. (2014) discussed that the most significant influence of horizontal stress is changes in the fracture mechanism in the immediate roof from bed bending failure to bed shear fracture which results in changing the main caving span. This phenomenon could be one of the reasons for fluctuation in the PAPs. Moreover, in this paper, the direction of principal horizontal in situ stress was deliberately taken parallel to the panel length which could impact on the induced stress pattern in turn. However, to achieve a realistic and clearer explanation, a 3-D simulation is necessary.

**CONCLUSIONS**

Results of numerical simulation to investigate the effect of some critical parameters on the peak abutment pressures with the help of UDEC software were presented. The following main conclusions are extracted from this study:

- The front peak abutment pressure is always higher than the rear one.

- Equivalent strength of immediate roof (EIRS), average spacing of bedding planes in the immediate roof strata and the vertical in situ stress have a direct relationship with the rear and front abutment pressures whereas this relationship is indirect for the immediate roof height and the extraction height.

- All recognized relationships are almost linearly with the exception of bedding planes spacing and the immediate roof height which are logarithmic growth and exponential decay, respectively.
The obtained results showed that there is no clear relationship between the horizontal in situ stress and the peak abutment pressures. This should be further scrutinized through 3-D simulations.

REFERENCES


Wilson, A H, 1986. The problems of strong roof beds and water bearing strata in the control of longwall faces. In *Proceedings of symposium on ground movement and control related to coal mining* (Organized by the Australian Institute of Mining and Metallurgy, Illawara Branch, University of Wollongong) (pp. 1-8).


HIGHWALL STABILITY IMPLICATIONS FROM LONGWALL MINING AT BROADMEADOW MINE

Dan Payne¹, Matt Martin, Bob Coutts, Dan Lynch

ABSTRACT: The Broadmeadow punch longwall coal mine in Central Queensland Australia has experienced significant highwall movement associated with the effect of longwall subsidence when the longwalls approach their final position close to the open cut highwall. In response to this movement Broadmeadow employed two types of broadscale highwall monitoring (radar and laser scanners) to provide full coverage measurement throughout three consecutive longwalls approaching the highwall. This was to attain a better understanding of the mechanism causing the movement and potentially enable prediction of instability. Results from the monitoring found the highwall is displaced to magnitudes unlike those typically measured in open-cut mining, and in direct contrast to typical longwall subsidence behaviour. This paper discusses the ground movements measured, monitoring methods used, safety measures established as well as theorising the failure mechanism. Recommendations are made for mine and pit designs for future punch longwall layouts. The paper shows how the movements measured are more aligned to some measurements made during stream valley closure studies previously presented at the International Conference on Ground Control in Mining (ICGCM) and challenges the mechanisms suggested by previous literature.

BACKGROUND

The mining of longwalls under or adjacent to large voids (eg stream valleys, escarpments or cliffs) is commonly associated with heavily vegetated or steep surface areas. In areas of extreme topographic variance, access for and/or to traditional survey pegs and stations or even new radar or laser technologies is limited. In addition, seldom have the longwall layouts aligned themselves parallel or perpendicular to the surface feature, making interpretation of any available surface movement data more complicated.

The punch longwall layout (Figure 1) is also quite uncommon (only undertaken at a small number of longwall mines in Australia) but creates the perfect configuration to enable a somewhat controlled study of the effect of longwall subsidence on a steeply dipping surface feature (an un-vegetated, evenly excavated open cut highwall). The relatively recently developed radar and laser scanning technology has also enabled near continuous, real time, sub millimetre monitoring of a full 500m wide x 100m high highwall and because punch longwall enables access and clear view the technology could be easily deployed as compared to the highly vegetated and variable topography of stream valleys.

Broadmeadow mine prepares the highwall for long term stability after open cast mining is completed. Slope and batter angles, bench configuration and pre-spilt blasts for the final strip are all designed with the punch longwall end use in mind. The bottom section of the highwall is the scaled to clean the highwall of any loose material. Highwall above the portal access pads is rock bolted as required before steel mesh sheets are draped over the highwall to cover all access pads. Drainage is prepared to direct water away from portal areas and prevent ponding.

The punch longwall layout makes use of abandoned open cut mine strips and drives gateroads directly into the seam at the base of the highwall with no requirement for main entries. The longwall is then retreated back towards the open cut and recovered just short of the highwall.

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leaving a safe barrier pillar. Broadmeadow coal mine has mined 11 longwall panels via this method, from two adjacent open cut strips.

![Figure 1: Punch Longwall Layout showing highwall and low wall and gateroad access at the base of the highwall (note the open cut has usually abandoned the pit by the time longwall mining occurs)](image)

The factor of safety of barrier pillars was always >4 using ALTS and therefore were designed for longterm stability, given the shallow overburden depth (90 m). Furthermore, longwall gateroad equipment limited the distance between the longwall and highwall. Highwall stability was also not expected to be an issue due to the high factor of safety and the fact that the seam dips inbye and the highwall is battered back to 65 degrees with two catch benches, so any subsidence was expected to pull the highwall toward the goaf not impacting the stability of the batter slope.

The general configuration of the highwall is approximately 95 m high from the floor of the coal to the natural surface, with wall angles of 65 degrees for the initial 50 m of highwall to the second bench and an angle of 38 degrees from the lower to upper bench. The overall slope angle from toe to crest is 34 degrees. This results in a horizontal distance of 140 m from crest to toe and that the longwall stopline is typically very close to directly under the crest at the natural surface. Figure 2 shows a photo of the gateroad accesses at the pit floor, the sumps between gateroads and the highwall bench slope configuration.

During normal longwall mining, the strata ahead of a longwall face strains towards the longwall goaf (Figure 3a and 3b). Inclinometer monitoring conducted adjacent to the Broadmeadow Mine longwall 11 panel during the start of the block confirmed that the direction of shear movement is in fact toward the centre of the void created by the longwall panel (Figure 4). This movement is caused by the tension generated when the overburden collapses into the goaf. The limit of ground movement on the surface ahead of the longwall face (to the sides and behind the goaf) is used to define the angle of draw. At Broadmeadow an angle of 19-26 degrees has been determined from LiDar (Airbourne Lasar Scanning) monitoring and traditional peg surveys of subsidence on the surface. LiDar Typical subsidence ground profiles as shown in Figure 3a have been experienced at Broadmeadow.
Figure 2 – Image of second Broadmeadow Pit (Radar monitoring units positioned on the low wall slope on the left hand side of the photo)

Figure 3a and 3b: Subsidence profile in section from Introduction to Longwall Mining and Subsidence (2007) and Systematic horizontal movements observed in flat terrain found in Mills (2001)

Figure 4: LW11 Inclinometer results showing displacement of shear towards the longwall goaf adjacent to the start of LW11 in confined ground (no adjacent voids) Mills (2016)
In addition, survey peg data has indicated that horizontal movement of points on the survey follow the traditional movement shown in Figure 3b; that being the case they are drawn toward the goaf as the longwall approaches and then move back in the opposite direction (the direction of retreat) as the longwall passes underneath and the surface settles back forward (lays down). It was this traditional understanding that led barrier pillar designers to believe the longwall subsidence would pull the highwall (which would be ahead of the final longwall position) toward the goaf and into an even more stable position.

However, the highwall movement and associated ground deformation observed at Broadmeadow as the longwall approached final position and was recovered, did not conform to either typical longwall subsidence profiles, or typical highwall movement, with values far exceeding any stability limits used in adjacent open cut mines (indicating the onset of failure). This outward movement, while not affecting the global stability of the highwall, destabilised local areas of the highwall around pre-existing defects/geological structure. A significant local wedge failure adjacent to the MG11 portal occurred during the LW11 recovery. This indicates that although the barrier pillar was overdesigned for vertical load, is may have been under designed for the horizontal push of the subsiding ground (overcoming the shear resistance along the bedding planes.

INITIAL OBSERVATIONS

Punch longwall mining at Broadmeadow mine commenced in 2005 from Ramp 4 of the Goonyella Riverside Coal Mine, mining longwall blocks from east to west toward the highwall and sequencing them north to south (Figure 5). Longwall blocks 1-5 were recovered from this first pit. The distances that the stoplines were designed from the highwall (barrier pillar width) varied with blocks 1-3 between 97m and 125m (shortest distance). While blocks 4 and 5 were 161m and 144m respectively. Blocks 6 and 7 were located between the two open cut pits and required conventional mains headings for access, with longwall 8 located at northern end of the second pit.

Concerns over geological structure at the northern end of the first pit meant that a buttress of blasted material was left in front of Longwall 1. After this panel’s extraction, numerous falls above the TG1 and MG1 pads were reported ripping the highwall mesh and a large section of sandstone failed into a sump below the highwall. It is now thought that these incidents may have been caused at least in part by longwall subsidence ground movements. Due to poor access little inspection of the benches was carried out and the highwall stability issues were not connected to longwall subsidence. As no monitoring was in place limited observations were made for the first few longwalls and the barrier pillars grew to 161m (measured shortest distance). However when LW8 was recovered, deformation of the highwall including lipping (horizontal shear and displacement resulting in over hang) of bedding surfaces, floor heaving at the base of the wall and cracking on the upper benches was observed. This visually indicated outward movement of the highwall. Longwalls 8-11 had a distance to the highwall toe of ~100m shortest distance and it was decided to take advantage of highwall monitoring techniques used in the adjacent open cut mines.
MONITORING

As well as visual inspections and a few pipe extensometers across cracks on the surface, GroundProbe SSR-XT radar was used for monitoring as the longwall neared the final stages of retreat for LW9 and LW10. This instrument has the capability to scan a distance of 30 m to 3500 m away from the radar setup, identifying failures to a resolution of 0.3 m x 0.3 m and 30.5 m x 30.5 m, respectively. At the reporting distance of 215 m for LW9, the integrated visual imaging system that resolves a 2 m x 2 m pixel was used. The accuracy of the measurement is sub 1mm and scans the entire area in about 13 min.

The Maptek Iite Sentry laser scanner was trialled alongside the GroundProbe radar on LW10 and used exclusively for LW11. Like the SSR-XT radar it has a sub millimetre accuracy and is capable of scanning the entire wall. However it can complete the scan in 6 min depending on the block size needing to be monitored. Another advantage of the laser technology is that it is spatially referenced allowing itinerant monitoring. This means the scanner can be shifted to a new location and maintain a correlation in the data before and after moving. Sentry can resume from any surveyed pillar and continue a complete database, while the Radar uses a stable reference point to determine the actual movement between the radar and the monitored surface. When the radar monitoring commenced on LW9, finding a stable reference point was difficult at first due to the global movement of the highwall. Both units are shown in Figure 6.

Figure 5: Broadmeadow Mine Layout showing underground workings and access from the first open pit for longwalls 1-5 and from the second pit for longwalls 8-11

Figure 6: Ground Probe Slope Stability Radar and the Maptek Sentry Laser Scanning System
Both of these techniques enabled real time graphical display of total displacement and rate of wall movement in the direction of the radar unit (which was positioned perpendicular across from the highwall on the low wall of the open cut strip). The supporting software also allowed real time triggers or warnings of increasing rate of movement and videos of displacement over time were able to be created. Results shown in this paper were only from the ISite Sentry used on LW11. Both monitoring units and techniques worked very well and determined that large scale highwall displacement of up to 1000 mm towards the open cut were occurring on a very similar scale and pattern on all 3 longwalls.

HIGHWALL MOVEMENT RESULTS

As mentioned, the configuration of punch longwall, (mining back to an established open cut highwall) provided the perfect opportunity to monitor ground movement as the entire highwall for 3 consecutive longwalls is visible and relatively unvegetated. Monitoring of the highwall during the recoveries of Longwall 9, 10 and 11 using both the Groundprobe slope stability radar and/or Maptek I-site Laser Scanner has provided high quality and accurate (sub 1mm) information over the entire area to perfectly describe the highwall movement in real time. Scanning from an upper horizon on the low wall (directly opposite and across the pit void from the moving highwall) and using a stable reference point, laser and radar technology easily achieve .4-.6 mm accuracy from that distance (~300 m) which has been confirmed in open cut for several years.

It was found that the first sign of highwall movement away from the longwall and towards the open pit was experienced when the longwall face was 300 m from the highwall toe. Outward movement of the highwall increased as the face retreated closer to the highwall. An early study of longwall 9 only (L. Clarkson, 2016) showed that the rate of highwall movement was directly correlated with the rate of longwall retreat. For all three longwalls, movement would continue over the entire highwall adjacent to the stopline until the longwall reached final position, and then show movement progressively from Tailgate to Maingate as the shields were recovered in sequence. At this stage the highwall movement virtually came to a stop in all three cases.

Variation in the magnitude of movement increased with distance up the stratigraphic section from the seam to surface and was divided into bands of movement by coal seams or other sedimentary layers such as the P-tuff claystone. These provided low friction interfaces for shear movement. The horizons where shear movement was observed with the inclinometer at the start of longwall 11 (although much lower magnitude and in the opposite direction) correlated exactly with the horizons of movement observed along the highwall. However, unlike ground behaviour at the start of the longwall panel, the entire highwall mobilised in the opposite direction (away from the longwall goaf).

Figure 7: LW11 Highwall total movement 19th of Jan to 30th March 2017 as a composite of scans every 13 minutes between those dates
There were three distinct zones of movement up the highwall (Figure 7).

- Directly above the coal seam being mined and below the P-Tuff layer, this section of highwall consisted of sandstone channel deposits and moved <100mm.
- Above the P-Tuff but below the GP5 coal seam, total movement of <300 mm was recorded. The GP5 was located just above the first bench which allowed observation of the lipping surface which formed along this interface (Figure 8).
- Above the GP5 coal seam to the upper bench, total movement of up to 1000 mm was recorded.
- Levee bank behind highwall moved towards the goaf as per the regular subsidence trough behaviour (including the slight difference in longwall take off position alignment with the highwall, ie. closer at the tailgate as shown in Figure 9).

![Figure 8: LW11 Highwall total movement and photo of lipping surface along the GP5](image)

In Figure 7, green areas display where the highwall has moved over 1000 mm towards the monitoring station (pit). As per the legend, blue areas indicate movement away from the scanner (toward the longwall goaf). The 2 blue patches on the lower face itself in Figure 7, are areas where material dropped off the wall leaving cavities behind. The upper dark blue wedge however shows true ground movement away from the scanner. The shape and location of the dark blue wedge clearly represents that the longwall is on a slight angle to the highwall (50 m closer at the tailgate) and undermines the very upper part of the slope at the tailgate end of the wall which is confirmed in a plan view in Figure 9.

The extents of the movement across the highwall quickly dissipated either side of the longwall gateroad entries. Total movement increased towards the centre of the longwall block (consistent with the rounded shape of subsidence contours and protection provided by pillars). Jointing appeared to provide a lateral boundary to movement as demonstrated by the vertical colour changes in Figure 10. These differential movement boundaries were observed in the highwall above the pillar between the belt and travel road portals. This is thought to have contributed to increased highwall instability which observed during the longwall recoveries. It is also interesting to note that instabilities occurred in the lower wall where the movement was the least, whereas the upper wall experienced much more movement. This is because the upper walls are comprised of weathered tertiary material (very soft) and didn’t have pronounced structure.
While it is assumed some deformation had occurred during longwall recoveries in the initial open pit (longwalls 1-7), no visual deformation was observed, and as such no monitoring program was undertaken. There are a number of differences between the first and second open cut pit.

- The initial three panels had a block width of 200 m, before Longwall width was extended to 320 m from LW4.
- Distance from the highwall for the initial open cut pit was greater for the full width panels than for the second.
- Top Coal caving was introduced on Longwall 8 which increased extraction height from 4 m to ~6 m. (although caving is not carried out for the last 450 m of the longwall to prepare for recovery)
Predicting Highwall Instability

Open cut coal mines have standard triggers for assessing highwall instability based on rates of movement. However, the magnitudes of movement generated by the longwall affect (1000mm total and rates up to 1.5mm/hr over +6 weeks) were unprecedented and the rate was controlled by longwall retreat rather than ground failure. Open cut coal highwall instability triggers are in the order of 5mm/hr and are the result of rock failing usually along a structure and show an increasing rate that can progress over as little as 40 minutes (local experience) to failure to as long as many weeks (and longer in open pit hard rock). Therefore although the data could potentially be reviewed every 13 min, interpretation required the geotechnical engineer to use judgment and temper increased rate of movement with increased rate of longwall retreat. Only increasing rates of movement during non-production time or extreme and accelerating rates of movement during steady production could be used as indicators. However, the ability to monitor the entire 500m wide and 100 m high highwall exposure and colour contour it, allowed identification of anomalous localised areas of movement and the triggering of additional protection measures against these.

Effect of Rockfall Mesh above the Portals

Punch longwall mining (accessing gateroads from the base of an open pit) increases the risk of mine inundation due to the low elevation and large catchment area. Therefore large levees are constructed around the open pit on the surface to protect from flooding from adjacent rivers and large sumps are constructed against the highwall between the headgate and tailgate portals to control rainfall in the local catchment of the pit. These sumps conveniently prevent exposure to the working area from rockfall hazards between headgate and tailgate however the portal areas remain exposed.

Due to the frequent access of men and materials through the portal entries under the 50m high highwall to the first catch bench, this portion of the wall is prepared with more stabilisation. The local highwall had been rehabilitated with rockbolts and then had rockfall mesh draped over it, to contain any local loose rocks from falling in the work area. In addition, substantial reinforced concrete portals are installed out to a distance of 15m from the highwall to allow covered access for men and materials and a 10 m exclusion is enforced adjacent to the portals themselves complete with a 2 m high rock bund to create a catch drain for any local rockfalls. This is standard for all punch longwall portal accesses and is independent of the results of this study.

Unfortunately due to the inability for the scanning to see through the draped mesh and the expansion and contraction of the wire mesh during day/night temperature changes, it made interpretation of wall movement in those local areas difficult to impossible (Figure 11) This was the case for both the GroundProbe SSR and the Maptek Sentry.

Figure 11: Effect of mesh draped over portals on radar and laser scanning monitoring
MECHANISM

In theorising the mechanism a review of typical longwall subsidence horizontal ground movement behaviour was undertaken, along with stream valley closure literature. When comparing this information with the full face movement results from the scanning, and looking at the behaviour in section. It was theorised that the forward movement of the highwall is primarily caused by the subsiding ground. It is proposed that as the strata lays down behind the longwall, a massive forward push of the ground occurs which is normally confined by hundreds of meters of solid ground, but in the case of being adjacent to an open cut void (or stream valley) shoves the bedded ground forward like a stacked deck of cards. With the maximum movement near the surface and decreasing downward due to leverage and frictional resistance from the weight of overburden and its proximity to the subsidence trough (Figure 12).

![Figure 12: Geological section above the LW11 stopline relative to the highwall showing relative surface ground movement directions](image)

Previous studies have been complicated by the difficulty in getting measurements and making observations, as well as the complex orientation of stream valleys to longwall layouts and mining direction. Figure 13, is taken from Hebblewhite (2001) which showed measurements of the same behaviour (away from the goaf) along the side of longwall panel subsidence and theorised the mechanism to potentially be horizontal stress, strong sandstones, gorge effect, vertical faults, or horizontal structure reactivation.

![Figure 13: Measurement of horizontal ground movement away from the longwall goaf and toward the stream valley (Hebblewhite 2001)](image)
There is evidence of another case study in Queensland where multiple pillars failed in a highwall mining scenario and rather than the highwall being pulled into the collapse, it was shoved out into the open cut with the same mechanism as proposed at Broadmeadow.

After properly measuring the magnitude of movement from longwall 9 a RocScience Phase 2 numerical model was built in an early study Clarkson (2016) to simulate the effect of longwall caving with the free face of the open cut excavation. Figure 13 shows the vectors of ground movement generated by the model which shows highwall movement away from the longwall. Although the model had difficulty simulating the effect of the push of the subsiding ground it did show movement of the ground away from the longwall and towards the highwall, albeit greater at the toe than the crest.

![Figure 14: Phase 2 numerical model attempting to simulate ground movement away from the longwall (Clarkson 2016)](image)

**DESIGN CONSIDERATIONS**

From an economic standpoint an underground planning engineer would seek to place the take-off position of the longwall as close to the highwall as possible. In the shallow depth of cover including a sloped bench scenario of punch longwall mining the width of the barrier pillar will never be limited by pillar stability. Design methods such as applying angle of draw from the crest of the highwall may be not be sufficient to account for highwall stability. While the global stability of the highwall was maintained at Broadmeadow, localised failures remobilising along joints or faults can be triggered. These may occur around pre-existing geological structures, cling-ons (material stuck on the wall over blastholes) or blast cracking. For LW11 movement was first observed with the longwall 300 m from the highwall toe, therefore if the longwall is to mine within 300 m of an open pit a number of controls should be considered:

- Feasibility studies should take advantage of technology to scan and map highwalls prior to planning portal locations and longwall stop positions to identify all potential structures that could be affected and specific controls put in place for those (or avoided).
- Catch benches and portal pads have space for adequate bunding against the slope toes to manage pit slope failures.
- Infrastructure placement on the highwall benches and pads allows for potential ground movement, where concrete portal entries are set further off the highwall.
- Cater for access and restricted access to catch benches.
- Ensure catch drains are accessible and regularly cleared to maintain capacity.

**CONCLUSIONS**
The punch longwall layout and the open cut slope stability monitoring technology provide a near perfect scenario for monitoring the effect of highwall movement due to an approaching longwall. The results in this paper add to the body of data that shows that longwall subsidence will push ground forward when adjacent to a void. Additional controls are required and can be very effective for working in close proximity to a highwall or void. Barrier pillar sizes in punch longwalls can be minimised with an understanding of the mechanism, appropriate design and controls.

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REFERENCES

Introduction to Longwall Mining and Subsidence, Mine Subsidence Engineering Consultants, August 2007
THE EXPERIENCE OF ADDCAR IN HIGHWALL MINING OPERATIONS

Paul Hartcher¹, Grant Case²

ABSTRACT: Within coming years, Highwall mining (HWM) as a technology, will be more frequently considered as a means to optimise resource recovery within a particular mining context and assessment of the risks/benefits associated with such a technology need to be fully addressed by those undertaking the assessment. The Addcar technology has been in the forefront of innovation and technological enhancements since its introduction in 1990. The experiences of the Addcar team, in Australia and the USA is that the requirements for successful utilisation of the system or its counterparts, the range of applications and the comparative capability of the relevant systems are often not fully understood and as a consequence opportunities are lost or projects commenced with unrealistic expectations. Through the use of explanatory examples and references, the authors seek to highlight those areas of critical nature that require specific focus when assessing the potential use of HWM technology and the range of applications such technology can be utilised in.

INTRODUCTION

Highwall mining (HWM) is a hybrid mining arrangement whereby underground mining methods are used in a surface environment using a combination of underground, surface and specialised equipment. Many in the Australian industry have broad views as to what constitutes HWM and relate past experiences and apply particular biases to strategic planning decisions that in some cases are technically outmoded and commercially incorrect.

The Addcar system has been operating the longest of any system (since 1990) and has mined more tonnes (approx. 120Mt) across a broader range of conditions and therefore is a suitable HWM system for benchmarking performance, establishment needs and resource assessment.

Discussion herein is therefore centred on Addcar experiences and learnings and further information on the specific nature of the Addcar and other HWM systems can be sought from respective suppliers.

This paper seeks to undertake comparative analysis between two distinct Eras (refer section 2 below) of HWM in Australia and notes operational differences between the USA and Australia where appropriate.

The intention herein is that potential users of HWM technologies can make educated judgements regarding recovery and production rates when assessing the full potential of any resource.

HISTORY

The Addcar system was first operated at the Boomer Mine in Fayette County, West Virginia in 1990 and since then has mined in excess of 120 million tonnes internationally with in excess of 21 million tonnes being mined in Australia. Other HWM variants have been developed but have not captured the market spread or achieved the consistency of results over such a wide range of operational conditions over the same time period.

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The Addcar system was initially introduced into Australia in 1995 and in 2014, the current owners purchased Addcar from Arch Coal and reintroduced the technology into Australia after a ten year hiatus. This period of inaction in Australia has resulted into two distinct timeframes of HWM operation nominated in this paper as:

- **Era 1**: 1995 to 2003, and
- **Era 2**: 2014 to present.

Consequential improvements to Guidance Technology, Resource Assessment and Geotechnical Design need to be applied to factual assessments of potential mining sites rather than reliance on various past outcomes with differing technologies and outdated design assumptions. In addition, by operating in differing international locales Addcar are in a unique position to relay information on factors affecting costs, efficiency and productivity that should be considerations for current review of operations and competitive analysis in Australia.

**A BRIEF COMPARISON OF HWM SYSTEMS**

The following table is for the purposes of illustrating the relevant potential differences between available technologies and is believed to be factual at the time of writing, each proponent should confirm current trends and system limitations at the time of assessment.

<table>
<thead>
<tr>
<th>Key Operational Factor</th>
<th>Addcar CM Based with conveyor coal clearance</th>
<th>Other CM Based Systems Auger based coal clearance</th>
<th>Auger Based Systems Auger coal cutting and coal clearance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth Capability</td>
<td>Subject to dip – 500 meters system capability, car conveyors are individually powered</td>
<td>Nominally 300 meters – car augers are powered from the surface</td>
<td>Nominally 100 meters</td>
</tr>
<tr>
<td>Dip Capability</td>
<td>Subject to dip – in excess of 15 degrees, have mined in excess of 20 degrees (<a href="#">Westmoreland Kemmerer Mine in Wyoming – 2018</a>)</td>
<td>Limited to 8 degrees</td>
<td>Limited to 8 degrees</td>
</tr>
<tr>
<td>Fines Generation</td>
<td>Typical underground fines generation using CM</td>
<td>Use of auger conveyor system generates higher proportion of fines than conveyor based coal clearance</td>
<td>Use of auger coal cutting plus auger coal clearance creates highest proportion of fines.</td>
</tr>
<tr>
<td>Guidance</td>
<td>3rd generation INS and 5th generation guidance system</td>
<td>Nil</td>
<td>Nil</td>
</tr>
</tbody>
</table>

**ASSESSMENT OF POTENTIAL**

It is generally the absence of key aspects as outlined below that result in poor assessment and incorrect decision making regarding the go ahead or otherwise for HWM.
Regional and local geological characteristics of the site must be determined in order to establish a base geological/geotechnical model leading to a Geotechnical Design for the mining layout.

Key data inputs include:

- Geological setting
  - coal measures features
  - regional folding and faulting
  - stress regime, igneous activity etc

- Sedimentology and Strata Conditions
  - nature and characteristics of overlying and underlying strata, lateral variability
  - Presence of water, coal seam aquifers, etc.
  - Floor trafficability
  - Gas permeability and desorption

- Structural features and Highwall Mapping
  - faults, jointing, seam rolls, measurements of orientation, dip etc.

- Seam characteristics
  - rank, brightness, cleating, shearing, dirt bands etc.
  - proposed seam cutting sections

Projects are justified on the basis of the resource recovered and the production rate, nothing impacts on both performance KPIs like “highwall surprises” and whilst most issues can be accommodated through appropriate management, it is impossible to meet targets when confronted with a feature that could be mapped and highlighted at planning/assessment stages.

Figure 2: An actual Highwall “Surprise”
HWM HISTORICAL KEY DATA

It is beneficial to have some understanding of the potential performance of HWM and whilst extremely difficult to readily transfer data from the USA to Australia, there is some useful comparative information that can be reviewed and assessed within the context of potential Australian projects.

Productivity

<table>
<thead>
<tr>
<th>MINING METHOD/LOCALE</th>
<th>Tons per Employee Hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Western Surface Mines</td>
<td>19.9</td>
</tr>
<tr>
<td>ADDCAR</td>
<td>10.0</td>
</tr>
<tr>
<td>Longwall Mines</td>
<td>4.3</td>
</tr>
<tr>
<td>Eastern Surface Mines</td>
<td>3.7</td>
</tr>
<tr>
<td>Non-Longwall Underground Mines</td>
<td>2.6</td>
</tr>
</tbody>
</table>

Table 2: Productivity (all US mines) by mining method - 2009

HWM productivity is a function of process, seam height and bench condition but nominally, output per man employed will be lower in Australia than the USA for comparative conditions due to OHS and IR requirements.

Sizing

The HWM system utilises a continuous miner to extract the coal and proponents should be aware that as a consequence the fines generation will be greater than that resulting from mining the same seam by open cut means and will be slightly less than conventional underground mining of the same seam due to the reduced shatter points in the coal clearance system.

The auger coal clearance systems result in an approximate increase of 20% in fines generation over the conveyor based coal clearance systems.

*When assessing the introduction of HWM, consideration should be given to the CHPP processing requirements and fines circuit throughput capacity.*

Safety

By far, statistically the biggest fatality safety risk to employees associated with HWM is Rock Falls from the highwall.

Other obvious risks relating to machine entrapment evolve due to inappropriate geotechnical design, non-adherence to the design or changed conditions in entries resulting in amendment to the design. The employee risk associated with such circumstance again relates to the highwall as any catastrophic subsidence event may lead to highwall instability.

Statistically, the most frequent cause of injury relates to machinery interaction.

Generally, safety statistics for HWM are equivalent or less than those achieved in surface mining.

Procedures, management plans, inspection and reporting regimes are required to address the exposures as noted and recent refinements to Permit to Mine (PTM), Entry Design reviews, highwall geotechnical inspection and monitoring, FOPs protection on equipment, use of safety berms have been instigated into daily and shiftly mining practice.

*Fundamental to the success of any HWM operation and the safety of the employees involved is the Geotechnical Design. The design is only as good as the data available*
and whilst the monitoring of conditions encountered are vital to continued design refinement, the critical phase of design is at the assessment stage prior to commitment.

**Resource recovery**

It is critical that the assessment phase of HWM potential studies utilises realistic resource recovery parameters.

A realistic boundary condition for entry depth consideration is actual depth which nominally will be 80% of planned depth so that for a defined resource block of 400 metres depth, the average achieved depth will be 320 metres.

Resource recovery is ultimately, primarily a function of seam height and depth of cover with additional analysis based on localised conditions being required. For thin seams with low height highwalls, 60% (by plan area) or higher can be achieved whereas for thicker seams and higher highwalls (depth of cover) the figure can be below 40%.

In thicker seams multiple passes can be undertaken but w/h ratios and rib stability issues eventuate and there is an economic consideration based on resultant coal recovery and required entry geometry.

> A conservative approach to resource recovery is appropriate but front end data capture and application remains the most reliable means of reliable entry design and therefore economic assessment.

**MINE PREPARATION (HIGHWALL, DRAINAGE, BENCH, SERVICES)**

Whilst the HWM system can mine varying dips, the Launch Vehicle (or its counterpart) requires the bench area to be prepared to enable efficient establishment, coal clearance and stockpiling and working area to allow traffic management and component handling.

**Pit layout requirements:**

- 5 degree cross-grade and dip maximums on bench floor
- 50 metre minimum pit width
- Drainage sump
- Stable highwall and low-wall
- Stand-off 8 metres

![Figure 3: Prepared Operating Bench with Berm and Drainage in Place](image)
Appropriate effort into sump design, low wall and highwall water drainage, berm construction, utility supply, traffic management and bench development leads to productivity as opposed to continued delays and lost coal if focus is not placed on this facet of the HWM operation.

**GUIDANCE**

Highwall mining (with drives as long as 500m) is an unmanned extraction system and all key data should be captured and relayed in an interpretive format to the operator on the surface.

The ADDCAR highwall Mining Guidance System (MGS) has been developed and proven in the field and since its introduction in 1996, there has not been a single incidence of mine structural failure or equipment entrapment attributable to navigational issues.

The primary purpose of the guidance system is to mine to the plan, leave a safe mine structure and protect machinery in the drive from localised strata failure by;

- Maintaining pillar width and prevent intersecting plunges (primary purpose)
- Maintaining a consistent floor cut horizon to control spillage and prevent miner and cars becoming bogged
- Maintaining roof/floor beams where required to prevent localised roof collapse and equipment damage or entrapment

The guidance system enables greater coal recovery and productivity as the factors of safety on the geotechnical design can be reduced. Current NSW guidelines requires an increased factor of safety for systems without guidance fitted.

ADDCAR’s Guidance System is the subject of ongoing innovation and enhancement and incorporates a number of specialized and integrated Proprietary components and software:

- A customised Inertial Navigation System (INS) which uses a computer, motion sensors (accelerometers) and rotation sensors (gyroscopes) to continuously calculate orientation of the miner (azimuth, pitch and roll). Addcar has the only current United States Department of Defence approval to use this technology.
- Odocam, which measures movement in or out along the drive, is a bespoke system that uses a computer, video camera and markers (effectively a ruler) to continuously determine and provide the precise position of the equipment in the drive.
- Gamma sensors positioned on the miner to sense the depth of coal in the roof or floor of a drive. These are bespoke crystal sensors which detect radiation given out by mineral materials (rock). They assist in controlling boom height / cutting within the desired constraints. *Successful use of the roof gammas is very much dependent on seam conditions and current preference is to utilise the floor gammas (more consistent over wider range f conditions) and use pre-set height limiters.*
- Vertical Reference Unit (Inclinometer) on the miner boom is lower performing INS unit which provides suitably accurate indication of the miner boom position.
- The guidance computer and operator interface (called MK4), is a bespoke solution which interfaces with the above sensors and determines and directs the operator via a graphical interface in the control cab.
The guidance system interfaces with the PLC control system of the launch and so enables interactions for automated control and control interventions under specific events.

The guidance system also maintains a history log of all drives for review and reference. This data is post processed to provide hole reports and profiles.

It should be noted the guidance system while suitably accurate is not a survey tool and does not provide the levels of accuracy typically seen or expected of surveys.

**GAS MANAGEMENT**

The legislation both in NSW and QLD is not specific to the spectrum of conditions as encountered in a HWM entry and the reintroduction of HWM brought with it a need to address historical misconceptions and to incorporate technological advancements with respect to gas management within entries.

Unlike UG mining where it is basically methane mixed with air and control is achieved by diluting the methane with air, there are multiple dimensions to flammability control when undertaking HWM. Manipulation of the oxygen concentration as well as the methane concentration can be achieved through the introduction of inert gas (nitrogen) and extraction rates.
Broad outcomes of the Investigation included:

- The effects of Nitrogen (N) inertisation and consequential low quantity of ventilation gas meant
  - At full production rates ~30% of gas goes into filling the void (based on peak coal production. Addcars would reduce this void slightly
  - Air velocity is in the order of 15 to 20cm/second (0.015m/s)
  - This means that gas make at the face might not show up on outbye sensors for a considerable period of time therefore the best indication for the explosibility of the environment is at the miner.

- HWM Methane Sources were broadly categorised into three main areas:
  1. **Cutter head** (accounts for 80 to 90% of emissions)
     - Will be proportional to the rate of extraction
     - Will increase if product size is reduced
     - Related to gas content and desorption rate
     - Pre-existing entries may reduce emissions, *needs to be assessed by site*
  2. **Conveyor**
     - Will be proportional to the rate of extraction
     - Increases with smaller product size
     - Will be proportional to the time coal spends on conveyor (length and speed)
     - Related to gas content and desorption rate
     - Pre-existing entries will reduce emissions
  3. **Rib / roof / floor and face** emissions
     - Will be proportional to the rate of advance
     - Will be proportional to area exposed
     - Related to gas content and desorption rate
     - More permeable coal will increase emissions
     - Pre-existing entries will reduce emissions

![Oxygen Related to Car Location Hole 8-10](image)

**Figure 6: Graphical representation of In-Field O monitoring results**

For the purposes of risk categorisation and monitoring, the three Zones were defined based on nominal entry length.

- **Zone 1**: At face (If face >100m from portal)
Oxygen (O) (3%) low as N discharge is at face
Methane (CH4) could be high right at cutter head if mixing not complete
no realistic probability of explosion

Outbye (approx. 100m from portal)
O still low (3 to 5%)
CH4 could be significantly higher than at cutter head due to rib and conveyor emission being added to cutting emissions (dependent upon length of ribs and conveyor)
Still no realistic probability of explosion

Zone 2: Outbye (80m down to 20m from portal)
Critical Zone – defined as the Transition Zone
O probably 12 to 18%
CH4 could be high (rare circumstance) due to rib and conveyor emission being added to cutting emissions but is likely to start declining due to dilution with air
Potential for explosive mixture in this area if controls not applied

Zone 3: Portal (20m down to portal)
O could be still 18 to 20%
CH4 rapidly declining due to dilution with air
Potential for explosive mixture in this area is rapidly declining due to rapid dilution of CH4

There is no clear legislation or guidelines for inertised highwall mining gas management but in conjunction with QLD Inspectorate the following trigger map (based on Coward’s Triangle) has been used to set alarm and trip levels when operating the Addcar HWM.

![Cowards Triangle](image)

Figure 7: Cowards triangle and operational buffer zones – applied to each monitoring zone

Trip levels are then set at a point where flammable gas is <50% of ignitable or fuel rich line when working in a low oxygen environment and are achieved by:

- Monitoring environment at the CM, either side of Transition zones and on LV deck
- Commence mining utilising a methane trip of 2.5% when oxygen >5%
- Automatically transitions when oxygen <5% to trip when methane >5.5%
- This system is not reliant on operator interaction.

Gas management in HWM entries in Australia is achieved through monitoring, control and injection by using the combination of the introduction of inert gas, the speed of extraction and the multi-point sensing throughout the entry.
Historically, there are quite a number of research papers that analyse and discuss highwall failures and HWM panel failures and this paper does not seek to elaborate beyond experiential outcomes and collaborative undertakings with nominated industry specialists.

A brief summary of historical failures in the Era 1 period in Australia highlights the impact of:

- **HWM Guidance** – a number of pillar failures, in Australia and the USA, occurred in the 1990s due to the narrowing of the pillar between entries as a consequence of the continuous miner tracking away from the entry alignment. Since the introduction (1996) and refinement (ongoing) of the HWM Guidance system, there has not been an occurrence of pillar failure due to entry misalignment with the Addcar system.

- **Geometrical Pillar Design** – a reliance on FoS as the principle design parameter and the exclusion of Barrier Pillars led to a number of “Cascading Pillar” failures. The use of Barrier Pillars in Panel Designs is now common practice.

- **Time Based Geometrical Decay** – insufficient allowance for the time based decay of entry geometry, especially in the presence of water and clays in surrounding strata contributed to post mining failures. Remains a critical issue in designing stability of pillars.

Current issues based on experiences (good and bad) since 2014 (Era 2) highlights the requirement to assess any potential HWM site against the full backdrop of proposed Entry Geometry and strata/ground/highwall conditions both at the time of mining and post mining. A common problem as encountered both in the USA and Australia is a simplistic over-reliance on geometrical determinations for entries based on singular application of accepted industry formulas.

Generally, data relating to strata, water, structure is poor or almost non-existent and as a consequence adverse outcomes can eventuate that are not HWM related but rather as a consequence of inadequate design due to the non-inclusion of localised conditions in any applied entry design.

Analysis of historical Australian highwall mining operations have highlighted the requirement to complete designs on the basis of pillar w/h ratios as well as FoS ratios, noting that localised geotechnical anomalies influence the w/h ratio significantly and must be accommodated in any conservative design.

Addcar have worked with various parties and currently undertake design based on an Upper Design Limit (UDL) with a minimum Factor of Safety (FoS) of 1.6 (as per US guidelines) and a minimum Width to Height ratio (w/h) of 1.2 based on the likelihood that pillars with a w/h ratio of <1 are particularly susceptible to catastrophic failure and the consequent need for a 20% buffer to allow for any significant structural defects that may be present in the coal, potential misalignment in the drives and/or unplanned increases in the cut height.

A Lower Design Limit (LDL) is based on a minimum FoS of 1.28, which is no more than 20% less than the recommended minimum FoS of 1.6, and as per the upper limit of the failed cases shown in the database, a minimum w/h ratio of 1.

References to geotechnical design issues in Era 2 are limited to those as experienced by the author(s). Four examples are referenced due to the specific outcomes and relevant contributory factors which are noted with advisory pointers for addressing in resource assessment stage and subsequent definition of operational requirements.

**Example 1:**
**Outcome:** Slippage failure of highwall face.

**Immediate Cause:** Cascading pillar failure

**Contributing Factors:** Deterioration of entry geometry over time due to the influence of water in strata and overlaying clays

**Remedial Action:** Altered entry/panel design to accommodate the eventual change in entry dimensions

**Resource Assessment:** A full assessment of highwall structure and surcharge plus strata and water ingress and potential for impact on entry geometry and probable deterioration phase.

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**Example 2:**

**Outcome:** Highwall debris fall, striking light vehicle.

**Immediate Cause:** “Ski jump” in highwall face deflected debris away from vertical trajectory and into light vehicle

**Contributing Factors:** Failure to maintain continuity of berms in bench

**Remedial Action:** Berm Construction

**Resource Assessment:** Most (if not all) highwalls will not be prepared with HWM in mind and therefore there is an increased probability of highwall debris falling. The need for a means to catch deflected debris must be incorporated into planning and mining permits. Berms can effectively be used to control rock fall hazards by creating a catch basin and providing an effective barrier to keep personnel out of the operational area; but they must be properly sized, located and maintained.

---

**Example 3:**

**Outcome:** Buried CM

**Immediate Cause:** Limestone roof settled on to the CM

**Contributing Factors:** Fireclay band beneath planned seam floor, not highlighted in data package, nil immediate drilling, and impact not incorporated into entry design – weighted load on limestone beam in roof as pillars pressed into clay leading to beam failure and entrapment of CM.
Example 4:

<table>
<thead>
<tr>
<th>Outcome</th>
<th>Buried CM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Cause</td>
<td>Rib failure</td>
</tr>
<tr>
<td>Contributing Factors:</td>
<td>The HWM entry design was based on two pass HWM to a height of 8/9 metres at a seam dip of nominally 20 degrees and achieved depth of cover of 200 plus metres. Alternate long/short entries and long entries had single pass to maximum depth (200 plus metres) and double pass to 100 plus metres. Decision made to try triple pass (12 metres high) resulted in pillar failure and highwall failure, CM was left in entry pending site visit by inspectorate, ribs failed in entry and CM was entrapped.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Remedial Action:</th>
<th>Attempted recovery of CM (failed)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resource Assessment:</td>
<td>Any decision to alter the agreed geotech/entry design must be undertaken in a processed manner incorporating all technical inputs and should not be made on the basis of &quot;operational experience&quot;.</td>
</tr>
</tbody>
</table>

**REGULATORY ENVIRONMENT**

**Consent to vary current plans:**

There are some inherent differences between Regulatory bodies in NSW and QLD and associated processes and attitudes to HWM that need to be understood in any assessment exercise and/or approvals process.

Statutory requirements associated with the technology, whilst incurring opinion and interpretation differences between sites, engineers and Inspectors leading to frustration over inconsistencies and delays, can generally be managed.

The more influential (to project commencement) regulatory problem is that associated with the applicable approvals/consent process. This is particularly problematic in NSW where the concept of a MOP variation (still current in QLD) has been replaced with convoluted requirements to apply to various regulatory entities, each of which has the potential to reopen the project to protestation from organisations focused on the cessation of mining and with extensive media savvy and clout.

*Therefore, any submission to regulatory authorities for future mining should include a provision around the use of HWM, this simple statement negates the variation process if in the future the operation wished to introduce HWM.*

In QLD and the USA, a process based on modifying current operation plans without reopening public access remains.

**THE (MYTHICAL) EXTRA STRIP**

Confusion exists at times within the industry as to what definitively constitutes Highwall Mining and direct and inappropriate comparisons are drawn between differing technologies (e.g. Addcar and Augers) and at times decisions are made that actually result in the sterilisation of resource. In addition, examples from past experiences are referenced and applied within a current project context, often resulting in incorrect conclusions.

Oft heard quote; HWM “destroys highwall” is based on a widely held view that by simply waiting there will be a turn in market conditions such that the economic limit of mining will alter sufficiently to allow surface mining to recommence and past decisions to undertake HWM have destroyed such opportunities.
Subject to appropriate assessment of the resource and suitability to HWM, a typical blocked HWM reserve should conservatively be able to achieve an average depth of 320 metres or more. At a conservative recovery basis of 45% this equates to the advancement of the highwall by approximately 144 metres or nominally three additional strips.

Any decision around the introduction of HWM versus waiting for the market needs to be based on a full economic analysis of where the market needs to be to achieve three additional strips versus the cash now.

The possible transition from surface extraction to punch longwalls is an obvious option but the capex requirements for equipment and panel development are not insignificant and there is a probable time delay in sourcing and establishing the longwall operations etc. One consideration is to lengthen the barrier between the surface and the take-off road and introduce HWM, obviously driven by resource and boundary conditions but if achievable, a barrier of 200/300 meters could be successfully HWM mined and early cash flow introduced into the economics of the project.

Of course a myriad of other factors associated with market availability and price, forex rates, anticipated time and confidence in market change occurring, need for sustaining cash, rehabilitation pressures and alternate options need to be considered, but the “watch and wait” position is rarely satisfactory as a stand-alone outcome.

CONCLUSIONS

HWM continues to be a realistic option to achieve the optimum utilisation of available resources, the assessment (feasibility) phase of any project should incorporate HWM into the analysis and leave open the possibility for the future.

Existing projects need to collate as much data as possible to confirm methodology, resource and production targets and alignment with the capability of the technology.

DISCLAIMER AND ACKNOWLEDGMENTS

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REFERENCES

Stan Michalek, Chief, Mine Waste and Geotechnical Engineering Division, Pittsburgh Safety and Health Technology Center, Ground Control at Surface Mines Highwall Hazards and Remediation.
R. Karl Zipf, Jr., Suresh Bhatt, Analysis of Practical Ground Control Issues in Highwall Mining, National Institute for Occupational Safety and Health, Pittsburgh Research Laboratory, Pittsburgh, Pennsylvania USA.
Yi Luo, Associate Professor of Mining Engineering, Coal and Energy Research Bureau, West Virginia University (Report Period: July 16, 2012 – July 15, 2013), Highwall Mining: Design Methodology, Safety and Suitability.

Johnny Sturgill, President Addcar Highwall Mining LLC, RMCMI Short Course - June 27, 2015
Maximizing Surface Mining Resources Using Highwall Mining.
THE DEVELOPMENT OF A NOVEL BACKFILLING TECHNOLOGY: CONCEPT AND BEHAVIOUR

Lihai Tan1, Ting Ren2, Xiaohan Yang3

ABSTRACT: A novel backfilling method for underground coal mines has been recently proposed at the University of Wollongong. Different from traditional backfilling technology (i.e. solid backfilling and paste backfilling), the main feature of this technology is that cementitious material with high water-to-solid ratio is directly pumped into the gob filled with coal reject aiming to fill the large number of voids. To verify the feasibility and potential advantages of this new technology compared to its counterparts, a series of compression tests have been conducted. A total of four cubic samples with the dimension of 300 mm and the height of 150 mm have been tested to better understand the effect of cementitious material on the compressive behaviour of the combined backfill. The experiment results show that the strength of combined material with confinement is significantly affected by the coal reject filling coefficient. Based on the experimental observations, the compressive behaviour of the combined backfill consisting of three typical stages, namely initial compacting stage, support improving stage and stable sedimentation stage has been determined.

INTRODUCTION

Environmental accountability is becoming a greater consideration for mining industries. Impacts on the environment and the local community have had a large effect on the feasibility and success of mining operations. It is therefore critical to minimize the quantity of waste production and develop a cost-effective method to dispose of the coal reject for coal operators. Backfilling technology has become a basic method to control ground subsidence and provide sufficient support for surrounding mine structures. It is also believed to be an effective disposal approach for the mine tailings which are space-consuming and harmful to the environment.

During the past decades, various backfilling methods have been developed and put into practical application to meet different geological conditions around the world. Among them, solid waste backfilling and paste backfilling technology are the two main methods widely accepted due to their high performance (Kesimal et al., 2005; Deng et al., 2016). A solid waste backfilling system always consists of transport, feeding and mine filling devices. To provide sufficient support to overlying strata, a build-in compaction mechanism is designed to compact the crushed solid waste material with a large content of voids aiming at improving the overall effectiveness (Deng et al., 2016). A high pressure compaction mechanism is therefore required, resulting in increased investment. Different from solid waste backfilling, the voids content can be significantly eliminated by the use of high density slurry for paste backfilling technology. However, the large investment and complex pumping system is regarded as the obvious drawback for paste backfilling technology (Chen et al., 2017; Emad et al., 2015).

Metropolitan Colliery is one of the earliest underground longwall coal mine in Australia with a history of more than 120 years. Mining subsidence caused by longwall extractions has presented great threats to the Colliery (Chang et al., 2009; Gillespie 2007; Cremonini et al.,

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To deal with the subsidence issue and abandoned rejects, Metropolitan Colliery has currently been operating the only coal reject emplacement project in Australia (Tarrant et al., 2012). The high density slurry consisting of Teetered Bed Separator (TBS) is pumped from the surface to underground because it is not permitted to commence backfill deployment prior to the completed longwall production (Moroney 2017). It can definitely ensure the slurry does not enter to the working face and cause some complications. However, it is therefore more difficult to maintain the effectiveness of backfilling. In particular, the void between the overlying strata and backfill material will still lead to unexpected subsidence of overburden.

Against this background, this paper presents a conceptual backfilling method for underground coal mines which has been recently proposed at the University of Wollongong. The novel backfilling technology is believed to be a cost-effective method to improve the backfilling effectiveness compared to its counterparts (Yu et al., 2019). A series of compression tests have been conducted to demonstrate its potential advantages.

**NOVEL BACKFILLING TECHNOLOGY**

Different from traditional backfilling technology (i.e. solid waste backfilling and paste backfilling), the main feature of this technology is that the cementitious material with high water-to-solid ratio is directly pumped into the gob filled with coal reject aiming to fill the large amount of unexpected voids.

Taking the pumping system in Metropolitan Colliery for example, the characters of this novel backfilling technology as shown in Figure 1 are summarized as follows:

**Figure 1: The proposed backfilling method based on existing pump system used in Metropolitan Colliery**

(1) The coal reject is pumped from the surface using the existing pumping system;

(2) Cementitious material is injected into the goaf after the backfilling of coal reject through the separate pump system underground.

(3) A portable pumping system can be used to transport cementitious material to the goaf;

(4) The cementitious material used herein has a high water-to-powder ratio.

**EXPERIMENTAL PROGRAMME**
Test specimens

A total of 20 specimens including four cubic specimens and 16 cylinder specimens have been prepared and tested at the High-bay lab in the University of Wollongong. The cylinder specimens with a diameter of 150 mm and height of 300 mm were tested to understand the compressive behaviour of unconfined samples, whereas, the other cubic samples with the dimension of 300 mm and height of 150 mm are tested to explore the effect of confinement provided by the steel box. The constant water-to-powder ratio (i.e. 2.0) was adopted for all specimens with cementitious material.

Coal reject filling coefficient $f_c$, the ratio between the filling volume of coal rejects and the total volume of sample is designed as the test variable in the present research. To determine the filling effect for goaf underground, in which the filling materials are in confined conditions, a series of confined compressive tests with four filling coefficients, namely 0.0, 0.33, 0.67, 1.0, were carried out.

Material properties

- Coal reject

To determine the size distribution of coal rejects used for the study, a sieve analysis was undertaken as per the following method: Australian Standard 1289.3.6.1: Soil classification tests— Determination of the particle size distribution of a soil—Standard method of analysis by sieving (Standard 1995). A total of 1002.39 g coal rejects were used for the test. As listed in Table 1, the sieve result shows that the particle size of most coal rejects fall within the range of 13.2mm to 19mm.

<table>
<thead>
<tr>
<th>Particle size (mm)</th>
<th>Sample weight(g)</th>
<th>Retained (%)</th>
<th>Pass (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.00</td>
<td>110.80</td>
<td>11.05</td>
<td>88.95</td>
</tr>
<tr>
<td>13.20</td>
<td>828.30</td>
<td>82.63</td>
<td>17.37</td>
</tr>
<tr>
<td>9.50</td>
<td>57.50</td>
<td>5.74</td>
<td>94.26</td>
</tr>
<tr>
<td>6.70</td>
<td>1.59</td>
<td>0.16</td>
<td>99.84</td>
</tr>
<tr>
<td>4.75</td>
<td>0.00</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td>&lt;4.75</td>
<td>4.20</td>
<td>0.42</td>
<td></td>
</tr>
</tbody>
</table>

- Cementitious material

FB200 pumpable grout with high water-to-powder ratio was provided by Minova Australia. It is a high yield cementitious powder that only requires the addition of water and a suitable placing machine. The data shown in Table 2 was obtained from the technical data of FB200. It is apparent that the compressive strength of FB200 is closely relevant to the water-to-powder ratio.

<table>
<thead>
<tr>
<th>Water-to-powder ratio</th>
<th>Compressive strength (MPa) at different ages</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 day</td>
</tr>
<tr>
<td>2:1</td>
<td>0.6</td>
</tr>
<tr>
<td>2.5:1</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Preparation of specimens

The preparation procedure of the samples included the following steps as shown in Figure 2:

- Welding the steel box with designed size and thickness;
Filling up the steel box with the required amount of coal reject and marking the position inside the steel box;
Mixing the cementitious material with water in a bucket;
Pouring the mixed slurry to the steel box until all voids have been filled up;
Covering the samples with plastic covers until the testing data (i.e. 7 days).

Instrumentation and testing procedure

For each specimen, two Linear Variable Displacement Transducers (LVDTs) were used to measure the axial deformation of the specimen. All confined compression tests were conducted using a 500 tons Avery compression testing machine with a constant displacement controlling rate of 1 mm/ min (Figure 3). Considering the requirement of practical application, all tests were terminated when the axial deformation exceed 15 mm, about 10% of the overall height of the sample.
EXPERIMENTAL RESULTS AND DISCUSSIONS

The typical failure mode of confined samples is presented in Figure 4. It is clear that the unconfined samples show brittle failure mode during the compression test. Whereas, the confined samples after test still show stability, which can be found from Figure 4 in which the outer steel box has been removed. It indicates that the backfilling material under the confined condition is much different from its counterpart without confinement.

![Figure 4: The final failure patterns of specimens in confined compressive tests](image)

The stress-strain curves for samples under confined compression are presented in Figure 5. For all samples, firstly, the stress increased very slowly with deformation increasing, indicating that their support capacity was very limited during this period. When the axial strain reached about 0.012, stress began to go up rapidly with higher curve slopes. Take the sample with $f_c = 0.67$ for example, the curve slope suddenly rose from 54.0 MPa to 264.1 MPa, which means that the sample could support higher loading with less deformation compared with it at the previous stage. Later on, the increase of stress slowed down with a lower and stable curve slope. It can be seen that the curve for pure grout without coal rejects was much flatter than others and the stress remained at a quite lower level, which indicates that pure grout backfilling without coal rejects may lead to a poor filling performance.

![Figure 5: Stress-strain curves for samples with different filling coefficients under confined compression](image)
In general, three stages can be concluded for filling materials with high-water content material in confined compressive tests. Take the sample with filling coefficient of 0.33 for example (Figure 6), the three stages can be described as follows:

- **Initial compacting stage**

  At this stage, the filling materials suffers an obvious subsidence under loadings at a quite low level. It shows that the filling material fails to resist deformation effectively at the beginning period. This may be regarded as a defect of high-water content filling material. As there is a high proportion of water within the filling material, some water that doesn’t combine well with cement may be squeezed out, resulting in an obvious initial deformation. Long curing time or proper weight ratios between water and grout may be helpful for controlling this problem.

- **Support improving stage**

  After being compacted, the filling materials begin to take effect at this stage. It is considered that it’s the best stage for the filling material where the most advantage of the filling material can be made.

- **Stable sedimentation stage**

  When the loading is beyond filling material’s support capability, the filling material will finally experience continued deformation. It means that overburden strata or ground foundations can cause serious subsidence even if the goaf has been filled with the high water content filling materials.

![Figure 6: Three typical stages for samples with filling coefficient of 0.33 under confined compression](image)

The best-fit curve as well as the equation between coal reject filling coefficient and confined compressive strength is shown in Figure 7. It’s evident that the there is a perfect exponential relationship between them as the determination coefficient for the fitting exponential function reaches as high as 0.9999.

Coal reject filling coefficient has a distinct effect on filling material's strength in different loading conditions. With the increase of filling coefficient, confined compressive strength of filling material increases exponentially while its unconfined compressive strength shows a decreasing trend. As the bonding strength between coal reject and grout is relatively low, failure is easy to appear in the bonding plane and coal rejects are then squeezed outward without confinement. Consequently the coal rejects have a negative effect on filling material's strength. In confined
compressive tests, as the lateral deformation is restricted, coal rejects are forced to bear axial loading. As a result, filling material strength is improved.

![Figure 7: The relationship between coal reject filling coefficient and confined strength](image)

$$s_f = 16.669 - 12.580e^{-1.558\varepsilon_f}, \quad R^2 = 0.9999$$

CONCLUSIONS

This paper presents a novel backfilling technology based on the current pumping system used in Metropolitan Colliery. The additional application of the pumpable grout which can be set up in the roadway outside the working area is believed to be a cost-effective and safe method to improve the load carrying capacity of backfilling materials. The preliminary test on the confined box tests show that the higher coal reject filling coefficient contributes to higher filling strength. When the coal reject filling coefficient reaches 1.0, namely the goaf was filled up with coal rejects before grouting, the best filling performance is expected.

In practical engineering, however, full filling with coal rejects may be difficult to put into practice with coal reject amount, budget or some other factors taken into consideration. Based on the positive exponential relationship between coal reject filling coefficient and filling strength, an optimal coal reject filling coefficient can be determined to meet the balance between filling performance and engineering factors.

ACKNOWLEDGMENTS

The authors gratefully acknowledge Metropolitan Colliery and Minova Australia for providing coal rejects and FB200 grout material for this experiment. Thanks to Joshua Carey for his valuable contribution to the experiment and Messrs Fernando Escribano and Ritchie Mclean for their support in the preparation of the experiment.

REFERENCES


DIGITAL TWIN BASED METHOD TO MONITOR AND OPTIMIZE BELT CONVEYOR MAINTENANCE AND OPERATION

Manfred Ziegler¹

ABSTRACT: The presentation is about a newly, developed method to monitor and optimize belt conveyors, based on a high precision reproduction of the conveyor behaviour using a digital twin. By this numerical reproduction and the continuously performed synchronization between calculation and measuring, the system achieves an unmatched transparency of the belt conveyor's actual condition. Besides precise information about the overall efficiency, the system delivers early warnings about any changings of the physical state of the conveyor. The generated data allows analysis about time, place and root cause of the changes as well as quantitative statements about the operational properties of the main components in terms of energy efficiency and achievable lifetime. This enables a significant reduction of cost and downtime on the monitored conveyors. In the long run, this approach has the potential to speed up the improvement of the main components. The presentation is interesting for operators, maintenance managers and suppliers of whole belt conveyors or single components.

INTRODUCTION

For many years, belt conveyors have been robust and reliable equipment for the economic transport of large quantities of bulk material, as they are produced in particular in mining. Conveyors with lengths of a few dozen meters to more than 20 km and transport capacities from a few hundred to 40,000 t per hour are state of the art today. Reliable design is no longer a problem thanks to standardized and internationally agreed calculation methods. However, on closer inspection, it can be seen that there are large differences in the availability of different systems and virtually no operator is able to calculate their actual specific transport costs per tonne of material and per km of transport distance. The reasons for this are, on the one hand, that each conveyor system is unique and therefore difficult to compare with others, on the other hand, that until today there is no recognized procedure to determine the necessary maintenance measures correctly and at the right time. Here is an enormous potential for optimization, which was the reason to develop the system described below.

PRIOR ART FOR EVALUATING THE EFFICIENCY OF A BELT CONVEYOR

The formula work for the design of a belt conveyor according to DIN 22101(2018) and CEMA (2014) is based on an estimation of the design and operational influencing factors and aims at an economically meaningful coverage of the expected requirements in the normal operation of the system. However, this formula work is unsuitable for making accurate predictions regarding the power demand and local stress of the components for specific operating conditions "below design", or for deriving the demand in other situations from a measured power demand in a particular operating situation. This will be briefly illustrated below:

The DIN formula for calculating the main resistance \( F_{HI} \) of a subsection \( i \) is as follows

\[
F_{HI,i} = l_i \times f_i \times g \times \left[m_{R,i}' \left( m_{G,i}' + m_{L,i}' \right) \times \cos \delta \right]
\]

wherein

\( F_{HI,i} \) Main resistance of section \( i \) [N]

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Geesman (2001) discussed a measuring rig and the horizontal moving resistances $F_H$ measured on different conveyor belts during operation as a function of the vertical load $F_V$. Figure 1 shows this measuring rig:

![Measuring rig](image)

Figure 1: Measuring rig according to determine the horizontal resistance of various conveyor belts

If one applies equation (1) to an installation section of the slope $\delta = 0$ and the length of a support roller spacing of this measuring stand, the theoretical main resistances shown in Figure 2 are obtained for different virtual friction factors $f$ compared to the actual main resistance measured for a particular belt (dashed line) and the main resistance calculated with the analytical calculation model described below (red solid line):

![Main resistance graph](image)

Figure 2: Main resistance calculated with different estimated $f$-values compared to measurements on a test rig according to Geesman (2001) and an analytical calculation method
It can be seen that with any choice of the virtual friction factor, only one loading situation can be calculated correctly. Conversely, it follows that the virtual friction factor itself depends on the load:

\[ f_i = f(m'_{L,i}) \]  

The specification of the \( f \)-value for a belt conveyor is thus only meaningful in connection with the mass flow for which this \( f \)-value applies. Even then, the evaluation of the efficiency of this belt conveyor on the basis of the \( f \)-value is highly questionable, since the mass of the belt and the idlers (\( m'_{G,i} \) and \( m'_{R,i} \)) are included in the calculation, although only the energetic expenditure to transport the material is of interest. In fact, the running resistance of the idlers is almost independent of the translational mass of the idlers, but is largely determined by the bearing size, the bearing clearance, the seal and the bearing and sealing grease used. Support rollers with very large translational mass, but otherwise the same design would thus lead to a same main resistance, but a mathematically lower \( f \)-value.

In summary, it can be said that the \( f \)-value is not meaningful for the assessment of the energetic efficiency of a belt conveyor and thus is not suitable for continuous monitoring.

**INFLUENCES ON THE MOVING RESISTANCE OF A BELT CONVEYOR**

The moving resistance of a belt conveyor depends on many influences. To distinguish are:

- constructive influences
- operational influences
- maintenance status of the system

In terms of design, the moving resistance is influenced by the choice of the troughing angle, the length of the center roller, roller spacing and diameter, belt width, belt weight and speed. These parameters are usually all known and usually constant over the life of the plant. In rare cases, the speed is adapted to the current load.

The operational influences come from the conveying process and relate to the height and temporary change of the load, lump size and distribution, specific density and internal damping of the material. These values can vary and are in part unknown.

The maintenance condition also has an influence on the moving resistance. Misaligned idler rigs, lack of cleaning, incorrectly adjusted scrapers and longitudinal seals not only lead to higher energy consumption, but also to misalignment and belt damage and generally to a shorter service life of the components (belt, idlers and pulleys).

Worth mentioning is also the influence of the main components "belt" and "idlers". In particular, the belt has a large influence (essentially thickness and characteristics of the running plate), which also changes with temperature and aging state. The selection of these components takes place initially for the construction of the system by the Original Equipment Manufacturer (OEM) - often according to the specifications of the end customer. In the replacement procurement often other makes are used, which can change the power requirements. Frequent phenomenon: an old belt with thin cover plates is replaced by a new one and then there are restrictions on the flow rate, as the installed engine power is no longer sufficient.

A closer look at this issue can be found in VDI Guideline 4459 (2011), from which the following table is taken:
Table 1: Relative influence of components and operating parameters for different conveyor types

<table>
<thead>
<tr>
<th>Capacity</th>
<th>Drives</th>
<th>Pulleys</th>
<th>Chutes</th>
<th>Conveyors</th>
<th>Belt-fitting station</th>
<th>Steering devices</th>
<th>Rollers</th>
<th>Scraper equipment</th>
<th>Belt-tensioning equipment</th>
<th>Loading signal</th>
<th>Friction angle</th>
<th>Ambient temperature</th>
<th>Optional: detection of belt splices</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 500 t/h</td>
<td>5</td>
<td>2</td>
<td>6</td>
<td>4</td>
<td>0</td>
<td>3</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>6</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>500 to 1000 t/h</td>
<td>5</td>
<td>2</td>
<td>6</td>
<td>4</td>
<td>0</td>
<td>3</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>6</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>&gt; 1000 t/h</td>
<td>5</td>
<td>2</td>
<td>6</td>
<td>4</td>
<td>0</td>
<td>3</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>6</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

CONCEPT OF THE "DIGITAL TWIN"

The basis of the "digital twin" is an analytical model of the belt system, which calculates the moving resistance along the belt from the current operating conditions, taking into account the structural properties. The following measured values are required for this:

- Effective power of all motors
- Belt tension at a known location
- Belt speed at a known location
- Loading signal at a known location
- Ambient temperature
- Optional: detection of belt splices in a known location

It is initially assumed that the properties of the components "conveyor belt" and "idlers" involved in the main resistance as well as the alignment state of the system are the same at all points. With optimum adaptation of the analytical model to the real belt conveyor and ideal homogeneity, the calculation of the belt force around the whole belt results in no deviation between the start and end value of the belt force. However, since the real system is not homogeneous, deviations arise which can be converted to a difference between the calculated and the measured motor power - even with ideal adaptation. The geometric mean of this difference over a sufficiently long period of time is a measure of the quality of the model and can be used to adapt the parameters of the analytical model via a suitable optimization algorithm. In reality, average deviations of well below 1% of the installed drive power are achieved. See Figure 3.
Figure 3: Illustration of the replication quality of the digital twin with changing load

The deviations themselves - in Figure 3 the green time curve - represent the inhomogeneity and thus also the optimization potential. From them, the information about the efficiency of each conveyor section and - if the passing of the belt splices is measured – of the individual belt pieces, from which the entire belt is composed can be obtained.

The analytical model is based on a mathematical-physical replication of the real conveyor. This also includes the temperature dependence of the belt properties and the heating and cooling behaviour of the belt.

The exact adaptation of the analytical model is made by skillful selection of the parameters of the functions, which describe the moving resistance as a function of the operating parameters "local load, speed, belt tension and temperature". Together with the data record that describes the structure of the conveyor, this condition parameter record maps the conveyor. Thus, the system can not only be simulated in the current operating state, but also in all possible operating conditions and for all ambient temperatures. As long as nothing changes in the physics of the system, this condition parameter set is valid and provides reliable results. Thus, in particular for a standardized operating state with a selected, uniform load and constant ambient temperature, a characteristic number can be formed which represents the specific energy requirement for transporting one ton of material over a horizontal transport distance of one kilometre. This "standardized Energy Performance Indicator" EnPI has the unit Wh / (t km) and allows the evaluation of the energy efficiency of a conveyor system over time, or in comparison to other systems.

If something changes in the physics of the conveyor (alignment state, degree of soiling, properties of the belts and idlers) or the quality of the measurement data (incorrect data acquisition), this is detected as a permanent exceeding of the specified tolerance value and an
automatic search for a new condition parameter set that meets the accuracy requirements is triggered. This is followed by a message to the responsible plant supervisor.

POSSIBLE STATEMENTS REGARDING THE OPERATING BEHAVIOUR AND THE MAINTENANCE OF A CONVEYOR SYSTEM

The EnPI is an index that provides a macroscopic overview of the condition of the conveyor system, acting as an early-warning system in the event of any deviation. With the current condition parameter set to which this EnPI applies, it is possible to make precise forecasts of the power requirements that will arise when another operating condition (load, speed, ambient temperature) occurs. With this it is possible, for example, to assess whether an increased production rate can be achieved with the installed motor power.

The simulation of the temperature behavior also predicts the development of the power requirement for the start-up of a conveyor system which has been standing for a relatively long time and therefore has adopted the ambient temperature. Thus, it is also possible to predict how the power requirement for restarting will develop with increasing downtime when the conveyor is shut down in cold weather.

The depiction of the conveyor section efficiencies shows which sections are conspicuous due to an increased energy demand and deserve special attention. Reasons for this can be:

- poor vertical alignment of the roller bearing stations \( \rightarrow \) increased power demand correlates approximately quadratically with the size of the load
- poor horizontal alignment of the idler roller stations \( \rightarrow \) increased power requirement is linearly dependent on the height of the load
- above-average accumulation of stiff idlers \( \rightarrow \) increased power requirement is independent of the size of the load
- Friction between the belt and stationary parts of the system \( \rightarrow \) if the misalignment is not due to asymmetrical loading, then an increased power requirement is independent of the size of the load, otherwise it depends on the load
- friction between conveyed material and stationary parts of the installation (for example, longitudinal seals) \( \rightarrow \) above a threshold increased power requirement, which increases with the size of the load

The conveyor sections with the lowest energy consumption can be used as a reference for which improvement potential is at least achievable through a general improvement of the conveyor state. This not only reduces the energy consumption, but also increases the maximum possible transport capacity with the currently installed motor power.

If the belt conveyor is equipped with a belt splice detection, the system can also determine the efficiencies of each belt. This gives the opportunity to prefer belts for replacement that have proven to be more energy-efficient, or to make concrete specifications to manufacturers in this regard.

For a specific application, the investment and belt efficiencies shown in Figure 4 were determined over a period of five months. Since this comes from a shiftable conveyor in an open pit mine, the range of conveyor section efficiencies is wide and significantly greater than what can be expected for stationary installations. The belt efficiencies between different manufacturers and belts of the same manufacturer from different batches differ considerably. This is consistent with previous studies by Ziegler (2009), where the running resistance of the belts was determined directly with the measuring rig described by Geesmann (2001).
Figure 4: Calculated efficiencies of the different system sections and belts used on the basis of measurements on a real conveyor system

The system allows the continuous display of the maximum delivery rate at which the overload condition occurs under the current conditions. This can be either the exceeding the installed power of the motor with the highest load, or the slip limit of the most critical drive pulley determined from the belt pretension, the wrapping angle, and the current friction value.

From the values for the local belt tension, the modulus of elasticity of the belts and the current motor power, the drive characteristics can be displayed relative to a reference drive and from this the cause of unequal power distribution can be derived. Possible causes for this can be:

Differences regarding

- Pulley diameter
- Gearbox ratio
- Belt elongation (especially for fabric belts)
- Filling levels (when using turbo couplings)
- Ballast resistors (when using slip ring motors)
- Supply voltage at the motor
- Motor characteristic
Since all measurement data and relevant calculated values are stored in a database, the operating stresses can be determined for each point of the installation, each belt and each component for any period of observation. If a defective component has to be replaced, then the achieved load spectrum can be determined and used for comparison with other components. This makes the overall profitability transparent for an optimal selection in replacement procurement.

If sufficient data is available, the substance of the currently installed components can be determined with respect to the statistically achievable load spectrum and it can be estimated from this, how many components have to be changed by a given time in the future. Thus, the maintenance effort can be planned easily and precisely.

**PERSPECTIVES FOR FURTHER DEVELOPMENT**

The possibilities offered by this approach cannot yet all be estimated. In any case, it will be feasible to model the behavior of the components in sections and quantitatively correctly from the inhomogeneity. This could be used to determine the running resistance of each belt as a function of load for each temperature, speed and aging condition. This will greatly boost the optimization of the components.

As experience grows, the analytical model can be further improved, allowing the useful division of the conveyor into individual sections with higher granularity (currently, the sensible section length is about 50 m / section). This allows a correspondingly more accurate localization of deviations.

The analysis of the specific causes of deviations can be used to identify typical signal patterns for these deviations and to implement an automatic plain text message for future occurrences of this kind.

In summary, the concept of the "digital twin" opens up unprecedented transparency for the operation and maintenance of conveyor systems. The basis for this is the correct description of the technical mechanics, the permanent recording of the most important operating parameters and the constant adjustment for the automatic adaptation of the model to reality. The evaluation capabilities developed on this basis can be continually improved and used to automatically generate accurate notifications of relevant changes to the persons responsible. The exclusive expertise of a few experts becomes accessible to every operator of conveyor systems using this system.

**REFERENCES**

VDI Guideline 4459, 2018. Design of energy-efficient Troughed Belt Conveyors (Beuth Verlag).
INTERMEDIATE DRIVE TECHNOLOGY AS A COST-SAVING SOLUTION FOR BELT CONVEYOR UPGRADES

Blake Burgeth¹

ABSTRACT: The demand on belt conveyors continues to increase. Availability and reliability are key factors for increased productivity. Linear booster drives for mining conveyors were initially used in the US in the 1980-90’s with marginal success. The primary issue of this technology was controlling and transferring the power to the belt. Voith has successfully applied linear booster drives to over 300 installations worldwide. The Voith TurboBelt “TT Linear Booster Drive”, is optimized in a complex engineering process to meet the customers’ targets. Linear booster drives can save 25% or more on both Capex and Opex costs. This paper highlights the advantages of the TT are Drive concept, the potential applications that can benefit from this type of conveyor design, and cover examples of this technology. The TT Drive solution significantly reduces the belt tensions, allowing the use of lower rated and lower cost belts. This can benefit new installations as well as extend the life and/or upgrade the capacity for existing conveyor installations.

Voith Turbo, a Group Division of Voith GmbH, is a specialist for intelligent drive solutions, systems, and comprehensive engineering services. Customers from highly diverse industries such as oil and gas, energy, rail and commercial vehicles, ship technology, mining, and mechanical engineering rely on the advanced technologies and solutions-driven expertise of Voith Turbo.

INTRODUCTION

Already for decades, Voith Turbo GmbH and Co. KG has gathered experience with drive components for mining and mineral processing equipment and bulk material handling conveyors. This experience, combined with the service and sales organization established all over the world, is expanded more and more to turn it into a competence center for drive solutions and optimizations of belt conveyor systems. Among others, the product portfolio in Figure 1 for belt conveyors comprises of TurboBelt Hese Pulleys, high tension turbine T design conveyor pulleys, TurboBelt Transfer Stations, TurboBelt Tension Stations, TurboBelt Storage Loop, and the TurboBelt TT Linear Booster Drive.

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Having that background, Voith was well prepared when RAG turned to Voith in 2015 to work out a comprehensive solution for the upcoming replacement of a 16-year-old steel cord belt.

**DRIVE UPGRADE WITH THE VOITH TURBOBELT TT LINEAR BOOSTER DRIVE**

The TurboBelt TT Linear Booster Drive (TT Drive) is a high-performance intermediate drive for belt conveyors. It extends the lifetime of belts in existing systems and influences the requirement to the belt rating considerably, by reducing the belt tensions, which results in a significant cost savings. Compared to the conventional intermediate tripper drive technology, the TT Drive does not require material transfer points. This reduces the strain on the belt and increases its lifetime by reduced top cover wear, less bending cycles on the belt splices, and eliminating potential belt rip risks due to errant tramp metal originating from the transfer point.

Eliminating belt transfer points also leads to considerable lower dust formation negating the requirement for any dust suppression or dust collection equipment. Moreover, systems with TT Drives require less space than conventional systems which is an important aspect in underground mining. On long conveyors, the TT Drive reduces the belt tensions so that a lower rated belt can be used saving enormous costs by using fabric belts instead of steel cord belts. Smaller drive components (motor, gearbox, pulleys, etc.) can be used too. Therefore, there is less space required for the conveyor drive or drives. On existing belt conveyor systems, the TT Drive allows for increasing the conveying capacity while retaining the same belt. In addition, the current drives, take-up winch, and pulleys may also be retained.

**Voith Turbobelt TT linear booster drive – functional principle**

Equipped with a head and tail station, the TT Drive is integrated into the actual belt conveyor so that the top run of the existing carrying belt rests on the top belt of the TT Drive belt. Power is transmitted linearly by means of friction between the TT Drive belt and the carrying belt. Belt tension forces are reduced through the length of the TT Drive resulting in a reduction of the maximum belt tensions.

**Voith Turbobelt TT linear booster drive – existing conveyor systems**

When planning an increase of the conveying capacity on an existing conveyor, or system of conveyors, drive power will need to be increased as long as the conveying cross sectional area is sufficient to achieve the target.

For a conventional drive solution this would mean that the existing belt needs to be replaced by a new one rated for the increased tensions. In addition, higher forces on the pulley, structural steel, and foundations at the drive station would require further modifications. An engineering review of the other high tension pulleys, low tension pulleys, and take-up would also be required potentially leading to other equipment, structural steel, and foundation improvements. This problem can be economically and timely, be solved by implementing TT Drives. The existing
head drive with its pulleys and the belt remain unchanged, while TT Drives can be installed and commissioned during a short shutdown period.

Increasing the conveying capacity and/or extension of belt the lifetime on existing large conveyor systems by means of TT Drives provides:

- Longer lifetime of belt compared to conventional intermediate drive technology
- Increase of conveying capacity on existing conveyor systems
- No need to replace the conveyor belt
- No need to convert the existing drive system and modifying steel structures and foundations

**Voith Turbobelt TT linear booster drive – new conveyor systems**

For conveyor systems with large elevation changes and high conveying capacities, high strength steel cord belts and large drive units are commonly required. Moreover, greater mine excavation with corresponding foundations for support structures must be prepared for the head drive(s). For conveyors utilizing TT Drives, the use of significantly smaller drive units and lower belt ratings are possible.

Long belt conveyors with high conveying capacity in new systems utilizing TT Drives provide:

- Utilization of lower rated belts
- Smaller drive units
- Smaller pulleys
- Reduced space requirements
- Optimized and standardized multi-motor drives

**CASE STUDY – DRIVE UPGRADE AND BELT PROJECT FOR CONVEYOR H2 AT PROSPER HANIEL**

The underground Prosper-Haniel coal mine in Bottrop belongs to RAG Deutsche Steinkohle and conveys 3,000,000 tons of hard coal per annum. There is a network of 141 km of underground tunnels extending to a depth of 1,150 m. The entire H conveyor line is designed for a conveying capacity of 2,000 t/h. The speed for conveyors H3 and H2 is 3.2 m/s. The material velocity was increased to 4.2 m/s by means of accelerating the conveyor in order to have a smoother material transfer to conveyor H1 which runs at a speed of 6 m/s. Figure 3 shows the flow diagram for the material transport system.
The H2 belt conveyor transports all the hard coal extracted from the longwall and the material from the drift excavation over a length of 1,270 m and has a lift of 186 m. The belting installed on conveyor H2 is a steel cord ST 5000 12/10. The last replacement of the belt on conveyor H2 was performed in 1999. In addition, the repair and maintenance demand on this steel cord belt has increased considerably over the past years.

Due to the very poor condition of the steel cord belt and as reliability couldn't be guaranteed any longer, the option to replace the conveyor H2 belt came up in winter, 2015. The premise for the belt replacement was to refrain from using the expensive ST 5000 steel cord belt and to use a fabric conveyor belt of PVG 2000/1. In order to be able to technically implement this measure with the significantly reduced belt strength (2,000 N/mm instead of 5,000 N/mm), the use of two TT Drives was required.

**Design and planning of the drive upgrade and belt replacement on conveyor H2**

The customer's key requirement was to be able to replace the ST 5000 steel cord belt with a considerably cheaper PVG 2000/1 fabric belt. The installation of two VoithTurboBelt TT Linear Booster Drives makes this possible by reducing belt tensions, thus reducing the required belt rating from 5,000 N/mm to 2,000 N/mm. Comparisons of the calculation results, with the most important parameters, are depicted in Table 1.

**Table 1: Calculation results for conveyor H2 (old - new)**

<table>
<thead>
<tr>
<th></th>
<th>ST 5000 12/10</th>
<th>PVG 2000/1 and 2 Voith TT Linear Booster Drives</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center Distance</td>
<td>1,270 m</td>
<td>1,270 m</td>
</tr>
<tr>
<td>Belt Width</td>
<td>1,200 mm</td>
<td>1,200 mm</td>
</tr>
<tr>
<td>Height of Lift</td>
<td>186 m</td>
<td>186 m</td>
</tr>
<tr>
<td>Mass Flow</td>
<td>2,000 th</td>
<td>2,000 th</td>
</tr>
<tr>
<td>Belt Rating</td>
<td>ST 5000</td>
<td>PVG 2000</td>
</tr>
<tr>
<td>Input Power Required</td>
<td>1.547 kW</td>
<td>1.486 kW</td>
</tr>
<tr>
<td><strong>Splitting of the Input Power</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Head Drive</td>
<td>Drive Pulley</td>
<td>2 x 400 kW</td>
</tr>
<tr>
<td></td>
<td>Drive Pulley</td>
<td>2 x 400 kW</td>
</tr>
<tr>
<td>TT Drive #1</td>
<td>Drive Pulley</td>
<td>- 1 x 250 kW</td>
</tr>
<tr>
<td></td>
<td>Drive Pulley</td>
<td>1 x 250 kW</td>
</tr>
<tr>
<td>TT Drive #2</td>
<td>Drive Pulley</td>
<td>- 1 x 250 kW</td>
</tr>
<tr>
<td></td>
<td>Drive Pulley</td>
<td>1 x 250 kW</td>
</tr>
</tbody>
</table>

As it can be gathered from the comparison of the conveyor calculations with and without TT Drives, the maximum belt tensions in the area of the discharge pulley are reduced from 646 kN to 240 kN through the use of TT Drives (Figure 4). This reduction of the maximum belt tension allows the use of a fabric conveyor belt with a rating of PVG 2000/1 while complying with all safety requirements. Moreover, it's beneficial for the 30-year-old drive and the conveyor structure which were reinforced and repeatedly repaired due to corrosion damage.

**Figure 4: Belt tension force diagram**
According to the results of the analysis, Voith developed the following solution for Conveyor H2:

- Install the first Voith TT Drive reducing the belt tension so that the lifetime will be extended until the next scheduled shutdown
- Install the second Voith TT Drive to further reduce the belt tension
- Replacement of the ST 5000 conveyor belt with a PVG 2000/1

**Cost savings – conveyor H2 drive upgrade and belt replacement project**

The installation of two 280 m long TT Drive systems reduced the belt tension forces and thus made it feasible to use a less expensive PVG 2000/1 fabric belt instead of an ST 5000 steel cord belt. Despite the additional PVG 1600/1 belts required for the intermediate TT Drive systems, there was a significant savings regarding belting costs. However, for the analysis of the total installed costs, the procurement of the two TT Drive systems must also be considered. According to the data in Table 2, the final result are a total savings of approximately 10%.

<table>
<thead>
<tr>
<th>Project Supply Differences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Belt</td>
</tr>
<tr>
<td>PVG 2000 instead of ST 5000</td>
</tr>
<tr>
<td>Belt Costs</td>
</tr>
<tr>
<td>60% Cost Savings</td>
</tr>
<tr>
<td>Belt Splice Costs</td>
</tr>
<tr>
<td>55% Cost Savings</td>
</tr>
<tr>
<td>Total Costs Including Supplying TT Drives</td>
</tr>
<tr>
<td>10% Overall Approximate Savings</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

Using a scheduled shutdown, and without additional downtime, Voith installed the first TT Linear Booster drive on Conveyor H2 at the Prosper-Haniel coal mine. With the installation of the first TT Drive complete, Voith enabled the Prosper-Haniel mine to continue using a severely damaged ST 5000 steel cord belt until the next scheduled shutdown thus avoiding unexpected downtime and corresponding loss of production.

During the next scheduled shutdown, Voith installed a second TT Drive on Conveyor H2. With that, the belt tension forces where further reduced allowing Prosper-Haniel to operate the belt conveyor with a less expensive PVG 2000/1 fabric belt. The project managers from RAG Deutsche Steinkohle, Ralf Dohle and Wolfgang Kosiuk, were fully satisfied with the Voith Project Management and Engineering Team and the results of the Voith TT Linear Booster Drives: “We have a double benefit of the solution offered by Voith. On the one hand, we could avoid unscheduled downtime caused by the damaged belt, and on the other hand, we now can use a fabric belt on the system instead of an expensive steel cord belt. We would opt again for Voith’s solution at any time.”

Completing the installation of the TT Drives provided the mining company a comprehensive solution and achieved an overall cost reduction and schedule of the project.
ASSESSMENT OF KEY PARAMETERS ON LOAD TRANSFER DURING DEVELOPMENT OF A SPIN-TO-STALL RESIN BOLT SYSTEM

Kent McTyer

ABSTRACT: Spin-to-stall resin bolting system originated in the late 1990’s at South Africa’s Goedehoop Colliery and has become accepted over recent years in Australian underground coal mines. The main benefit touted is a simplified installation procedure that can lead to both reduced bolting cycle times and more consistent installation quality. These benefits are consistent with mining industry requirements of greater overall efficiencies – especially in mine roadway development rates.

Traditional pre-tensioned resin bolting practice has employed a bolt-rotation stoppage, or hold time, to allow the fast-set resin anchor to fully cure before re-starting rotation to pre-tension the bolt. Spin-to-stall bolting diverges from traditional methods by continuing to apply rotation until the bolt stalls. Continued application of rotation breaks the nut pin allowing the nut to run up the thread to produce pre-tension. While the simplified installation procedure is beneficial, concerns exist over the reduction in resin mixing time – especially at the top of the drill hole. Specifically, whether there is sufficient time for the resin to properly mix and attain full strength. Without full strength the resin anchor at the top of the bolt has reduced ability to transfer load – reducing the resin bolts effective length.

This paper discusses some key resin mixing parameters and their effect on load transfer. These parameters were tested during the development of a spin-to-stall resin bolting system. The test methods simulate underground spin-to-stall installation practice. This assessment technique is thorough and provides industry with greater confidence when evaluating the performance of a spin-to-stall resin bolting system.

INTRODUCTION

Roof bolting is the dominant primary roof support method used in underground coal mines. They have superior geotechnical applicability and performance in the often-variable strata sequences present in coal mine roofs. Roof bolting is suitable for skin support, suspension of weak or broken roof from more competent strata horizons, beam-building in thick sequences of laminated strata, and for keying of fractured and blocky rock masses (Mark 2000).

The evolution of roof bolting has led to most Australian coal mines using full resin-encapsulated torque-tensioned roof bolts. Bolts employ a fast-setting resin top anchor, a slow-setting resin anchor along the lower length of the bar, and a bottom anchor consisting of the roof plate and tensioned nut. These three elements allow for load from ground movement to be transferred to the steel bar, thereby resisting further ground movement.

Two installation practices, spin-and-hold, and spin-to-stall, are commonly employed. Both involve pushing and rotating the bolt through the resin capsule and spinning the bolt at the top of the drill hole. The more common -- and first-developed spin-and-hold method -- employs a bolt rotation hold time of 15 to 20 secs. This permits full hardening of the upper fast-set resin before application of torque-tension via the nut as shown in Figure 1. Adherence to the specified hold time has long been believed to form long polymer chains and, consequently, maximise the

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University of Wollongong, February 2019
strength of the upper resin anchor. However, adhering to the correct resin hold time at most mines is human-controlled, and not always possible due to competing production requirements. Failure to adhere to the recommended installation procedure can result in both over- or under-spinning of the resin, or incorrect timing of torque-tension, both of which will reduce the effectiveness of the roof bolt to transfer load. The recognition of the difficulties in achieving consistent installation practice has driven alternative resin-bolt installation practices such as the DSI PEAK System – a single-speed resin system, and spin-to-stall resin bolting.

Spin-to-stall resin bolting aims to simplify the installation procedure by removing the requirement of a bolt rotation hold-time. The installation practice allows the bolting operator to continue spinning the bolt through the resin until the resin rapidly hardens and binds the bolt, causing it to stall. Continued application of torque, via rotation, shears the pin in the nut. This bolter-delivered rotation then runs the nut up the bolt to develop pre-tension of the bolt as shown in Figure 2. The installation stages of spin-to-stall can also be identified via instrumentation fitted to the drill rig shown in Figure 3.
Spin-to-stall is a resin bolting system that has been in use since the late 1990s in South African coal mines (Altounyan, et al., 2003). In Australia, the first documented trials were at Grasstree mine in 2011 (Emery, et al., 2015), with increasing market uptake occurring since. Greater acceptance of the spin-to-stall installation procedure has been hindered by both the sensitivity of the system to variation in drill motor torque performance (both across an individual mine and across industry), and uncertainty as to the damage caused by continued rotation of the bolt through the resin polymerisation phase. Another concern is the use of a faster-gelling resin in the top portion of the resin capsule to promote more rapid resin polymerisation and bolt stall. The reduction in resin gel-time also reduces the required mixing time. Note that it is the completeness of the blending of the catalyst and mastic components that uniformly polymerises the resin and ensures it attains full strength.

The primary purpose of this paper is to detail the method and results of a comprehensive validation procedure and discuss some of the lessons learnt during the development of a spin-to-stall resin bolt system. The focus is on two parameters. The first is the influence of pin-nut breakout torque, and the second is the bolt feed rate during installation. Both parameters are systematically varied, with the aim being to increase the resin mixing time when the bolt is at the top of the drill hole. The posited theory being that increasing resin mixing time will result in both higher resin strength and better load transfer performance.

**BOLT INSTALLATION AND TEST TECHNIQUES**

The bolt installation and test techniques were developed over several years to evaluate all resin-bolt systems used in soft and hard rock mines and in tunnelling operations. The test procedure was first detailed in Evans (2016). It provides a comprehensive data set for each test bolt. Installation parameters were fully measured using instrumentation on the drill rig illustrated in Figure 3. Load transfer tests simulate underground short embedment pull-testing (SEPT), and the resin-failure interface and resin-anchor defects were evaluated. Overall, the methodology provides a means to both simulate underground spin-to-stall installation practice, and correlate controlled surface and laboratory-based test results with underground test data.
Drilling and installation of resin-anchored bolts was conducted using the DSI coal mine equivalent Sandvik D0100 drill rig shown in Figure 4. Test bolts were anchored into 101.7mm outer diameter, 93.7mm inner diameter steel cylinders with a length of 1800 mm. The steel cylinders were filled with approximately 70 MPa cementitious grout. A 25 mm diameter PVC tube was used to form a pilot hole to ensure the drill hole remained located in the centre of the cylinder. 28 mm diameter semi-spade drill bits were used to drill the hole before resin-bolt installation. The drilled holes simulated underground drilling conditions in a homogenous strata-type.

Figure 4: Drill rig facility at DSI Underground

Instrumentation during bolt installation recorded bolt travel (displacement) and revolutions per minute (rpm). In addition, a donut-shaped hydraulic load-cell was placed between the roof bolt
plate and the drill mast head plate to estimate the retained bolt pre-tension shown in Figure 4. The following consumables were used during testing:

- DSI AXR-profile bar with a nominal core diameter of 21.7 mm and major diameter of 24.7 mm. AXR bar has a typical yield strength of 240 kN, and Ultimate Tensile Strength (UTS) of 340 kN, with 15% elongation (measured after fracture).
- 36AF nuts with high (115-149 Nm) and super high (216-298 Nm) break-out pins.
- 150mm diameter, 5 mm thick star plate, anti-friction washer, and dome ball.
- RocBolt Australia RQ120024STS resin capsules (TORQ). The upper 600 mm of the resin capsule was extra-fast gel-time with the lower 600 mm slow gel-time. Catalyst proportion is approximately 10% of the resin volume.

The three test sample groups are shown in Table 1.

Table 1: Test sample groups, pin-nut torque and bolt feed rate

<table>
<thead>
<tr>
<th>Test Sample Group Name</th>
<th>Pin-Nut Breakout Torque</th>
<th>Bolt Feed Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Breakout, Standard Feed (HB/SF)</td>
<td>115-149 Nm</td>
<td>150mm/sec</td>
</tr>
<tr>
<td>Super High Breakout, Standard Feed (SHB/SF)</td>
<td>216-298 Nm</td>
<td>150mm/sec</td>
</tr>
<tr>
<td>High Breakout, High Feed (HB/HF)</td>
<td>115-149 Nm</td>
<td>200mm/sec</td>
</tr>
</tbody>
</table>

Following test completion, two 300 mm-long samples were taken from each complete bolt installation shown in Figure 5. The 300 mm-long samples were taken from all horizons to provide a load transfer profile from 50 mm to 1400 mm from the top of the bolt.

Figure 5: Spin-to-stall test samples

The samples were cured for 7 days, then each 300 mm-long sample was pull-tested on the DSI Universal Test Machine (UTM) until either: (a) complete bond failure, or (b) the load attained 300 kN. Tests were discontinued after 300 kN to avoid bar breakage. A plunge extensometer was positioned on the top of the sample bolts and pull-tests were video recorded as shown in Figure 6. The plunge extensometer provides a more accurate measure of bolt displacement compared with readings from the UTM cross-head, which contains both take-up of slack in the machine and elongation of the bar (post-yield).
Samples were sectioned and inspected after pull-testing. The inspections quantified the proportions of gloving and uncured resin for each 50 mm increment shown in Figure 7. In addition, each observed resin-anchor failure mode was classified as either (a) resin-to-rock, (b) shear within the resin annulus, or (c) resin-to-bolt as shown Figure 7. This resin-anchor failure mode information provides context for the load-transfer results.
TEST RESULTS AND DISCUSSION

Sixty-two bolts were tested during development of the DSI TORQ resin system. Tests compared various combinations of resin components, bolt profile, pin-nut break-out torque, and installation parameters such as feed rate and rotation speed. This report details tests used to refine the spin-to-stall resin bolt system and installation procedure. At this stage the DSI AXR bar profile and resin formulation had been selected based on best and most consistent performance. Thus, the tests discussed in this report examine two parameters: (a) pin-nut break-out, and (b) feed rate. The three test sample groups listed in Table 1 investigate mixing time at the top of the drill hole and how this affects load transfer and resin mixing properties.

Installation Parameters

Drill rig instrumentation (see Figure 3 as an example) was used to record and calculate average installation parameters for the three test sample groups. Six high breakout standard feed – HB/SF, four super high breakout standard feed – SHB/SF, and four high breakout high feed – HB/HF bolts were installed. Bolts fitted with high breakout pins (HB/SF and HB/HF) spent 3 to 4 sec mixing at the top, while those fitted with the super high breakout pin (SHB/SF) mix the resin for 5-6 sec at the top of the hole. A slight (0.4 sec) increase in mixing time at the top of the hole was found in HB/HF bolts compared with the HB/SF samples. Total installation time decreased from HB/SF (14.1 sec) to HB/HF (13.1 sec). Pre-tension was 10 to 13% lower for the HB/HF sample group compared with the HB/SF and SHB/SF sample groups listed in Table 2.

Table 2: Installation parameter and pre-tension averages

<table>
<thead>
<tr>
<th>Test Sample Name</th>
<th>Group Name</th>
<th>Test Sample Group Name</th>
<th>(n)</th>
<th>Time to Top of Drill Hole (sec)</th>
<th>Insertion Rate (mm/sec)</th>
<th>Spin Time at Top of Hole (sec)</th>
<th>Time to Nut Break-out (sec)</th>
<th>Total Installation Time (sec)</th>
<th>Pre-Tension (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB/SF (6)</td>
<td></td>
<td></td>
<td>6</td>
<td>8</td>
<td>150</td>
<td>3.4</td>
<td>11.4</td>
<td>14.1</td>
<td>58</td>
</tr>
<tr>
<td>SHB/SF (4)</td>
<td></td>
<td></td>
<td>4</td>
<td>8</td>
<td>150</td>
<td>5.5</td>
<td>13.5</td>
<td>15.5</td>
<td>60</td>
</tr>
<tr>
<td>HB/HF (4)</td>
<td></td>
<td></td>
<td>4</td>
<td>6</td>
<td>200</td>
<td>3.8</td>
<td>9.8</td>
<td>13.1</td>
<td>52</td>
</tr>
</tbody>
</table>

Load Transfer

The three sample groups were tested to compare peak pull-out load shown in Figure 8a. Individual samples were averaged when horizons overlapped. All tests used resin from the same box/batch.

The SHB/SF and HB/HF samples returned a similar load transfer trend over the tested bolt length. The HB/SF samples saw a consistently lower peak load. The returned load is similar for the three sample groups from 850 to 1400 mm from the top of the bolt. This indicates that the slow-setting resin portion of the bolt was less affected by changes to feed rate or total mixing time.

All three test sample groups showed a reduction in peak load transfer from 50 to 350 mm from the top of the bolt. This reduction may be explained by several factors including reduced mixing time at the top of the resin column, and increased concentration of gloving – explained further in a following report section. The 50 to 350 mm region of the bolt showed different resin mixing times for the three sample groups. The results indicate that greater mixing time in SHB/SF and HB/HF tests resulted in higher peak load transfer.

The greatest difference in peak load between sample groups was found from 350 to 850 mm from the top of the drill hole. This segment of the bolt consists of fast-setting resin and is the region believed to be the first to set and cause binding of the bolt. It is also the section of the
resin column subject to torsional forces during bolt stall and then axial forces during pretensioning. The load transfer results indicate that the HB/SF sample group was more negatively affected. This could be due to bolt stall and tensioning occurring when the resin is in a more critical phase of polymerisation. It is also possible that the SHB/SF and HB/HF samples had already passed this polymerisation phase and are less negatively affected by the bolt stall and tensioning forces.

![Figure 8: (a) pull-out force by depth along the bolt (left), and (b) pull-out load and displacement for the three test specifications](image)

It is generally accepted that the more complete the resin polymerisation, via adequate bolt rotation mixing time, the stronger the resin. The sample groups that had greater average mixing time in the top half of the resin column were the SHB/SF and, to a lesser extent, HB/HF. The results indicate that these two sample groups returned better load transfer. This is believed to be due to more complete mixing of the upper resin column, resulting in stronger resin that is better able to retain integrity and bond strength at the time when torsional and tensile forces occur.
Bolt and Resin Stiffness

Resin-anchor performance was investigated by measuring both the bolt displacement at the top of the bolt with a plunge extensometer, and the load recorded by the UTM as shown in Figure 6. This method isolates bolt and resin movement through the grout sample, and removes the additional displacement caused by both clamp settlement and bolt elongation, which was recorded by the UTM crosshead displacement. It is therefore a useful method to evaluate the performance of different bolt profiles and resin types. Results of the three test sample groups were averaged at 25kN intervals and plotted in Figure 8b.

As expected, stiffness of the three sample groups were essentially the same because the same resin and bar was used. However, the SHB/SF sample group was consistently stiffer from 50 kN to the limit of the tests at 300 kN. Again, the increased stiffness is believed to be a result of the additional mixing time prior to stall and pin-nut breakout. This produces a more uniformly mixed and polymerised resin with higher strength. Higher resin strength is believed to cause stronger mechanical interlock at the bolt-resin and resin-rock interface, and greater resistance to resin shear in the annulus. Thus, bolt displacement is reduced for a given load.

Resin Failure Mode

Bolt resin anchors were observed after load testing by removing the steel-encased grout cylinder from the resin-encapsulated bolt. Resin interface failure mode was defined as either: (a) resin to bolt; (b) resin annulus shear; or (c) resin to rock. Each 50mm length was logged, with averages for each of the three test sample groups calculated as shown in Figure 9. Common understanding is that failure at either the bolt-resin, or shear within the resin annulus, will result in higher load transfer than failure at the resin-rock interface. However, this assumes both a strata-type that is significantly weaker than the bolt-resin bond strength, and shear strength of the resin annulus. It also assumes equal resin strength along the bolt length.

The test results show that failure at the resin-rock interface is commonly found where load transfer is high. This finding may be due to both the high load at failure caused by the high grout strength, and the drill hole profile formed by the semi-spade drill bits (which produce minimal rifling of the drill hole wall). Regardless, it is the common occurrence of resin failure in the upper 350mm by both shear of the annulus, and at the bolt resin interface where load transfer is lower, that suggests resin mixing is a key determinant of the mode of resin failure. Again, complete polymerisation of the resin will produce a stronger resin more able to resist the loads occurring during a pull-out test. Conversely, a less polymerised resin will be weaker and therefore more likely to fail both in shear, and at the bolt-resin interface. This incomplete mixing is believed to be the reason for the concentration of resin-bolt and resin-shear failures at the top of bolts where total mixing time is less than lower parts of the bolt. However, it is also noted that weak resin may also fail at the resin-rock interface. This may be the cause of the less-than-conclusive relationship between resin failure mode and load transfer.
Gloving and Uncured Resin

Bolt samples were investigated for the presence of gloving and uncured resin following pull-out load testing. Proportions of gloving and uncured resin were recorded every 50 mm as shown Figure 10a.

Uncured resin was found on one sample from 100-200 mm from the top of the bolt caused by gloving and non-rupture of the catalyst compartment as shown in Figure 10b. To avoid adversely skewing the results this sample was removed from the HB/SF sample group. No other uncured resin was observed. Gloving was pre-dominantly found in the upper 200 mm of each of the three test sample groups. This was expected because the capsule film will tend to wrap and collect around the top of the bolt during insertion to the top of the drill hole. Another concentration of gloving was found in the HB/HF sample group at 500-600 mm. While the cause of this gloving concentration is not understood, it reflects similar findings to McTyer (2015). Low proportions of gloving are also found scattered across the length of the bolts, but was typically only a fraction of the bolt circumference. These low-level gloving proportions are judged to be the background level for bolts installed using resin capsules and, based on the results, are not considered detrimental to load transfer when the underlying resin annulus is fully cured.
SUMMARY

DSI Underground has developed a systematic methodology for the assessment of resin-anchored bolts commonly used in coal and metal underground mines and tunnelling operations. The method simulates underground installations using drill rig instrumentation, pull-out load testing and observational methods of the resin annulus to comprehensively evaluate each test bolt. Further, the range and number of tests performed on each bolt ensures that any conclusions are supported by multiple evidence sources.

A key conclusion of the spin-to-stall tests is that increasing resin mixing time at the top of the resin column is pivotal to ensuring the best possible load transfer performance of the resin anchor. Traditionally, this has been achieved by spin-and-hold resin that requires a 12 to 14 sec spin time, of which, typically 4 to 6 sec is spent mixing resin at the top of the resin column at the top of the drill hole. However, the change in installation to a spin-to-stall practice involves a faster gelling resin in the top portion of the resin capsule. This promotes more rapid resin gelation and stalling of the bolt. Yet, the reduction in resin gel-time can also reduce mixing time. Noting that it is the completeness of the blending of the catalyst and mastic via bolt rotation that uniformly polymerises the resin, ensuring the resin attains its full strength. Thus, to ensure
maximum mixing is achieved before stall, this study changed the installation procedure to evaluate both an increased bolt feed rate, and a change to a higher breakout pin-nut. These modifications attempt to increase the top of drill hole mixing time.

Three test sample groups were evaluated: (a) 115-149 Nm nut breakout pin and 150 mm/sec feed rate – HB/SF, (b) 216-298 Nm nut breakout pin and 150 mm/sec feed rate – SHB/SF, and (c) 115-149 Nm nut breakout pin and 200 mm/sec bolt feed rate – HB/HF. The changes to higher nut breakout torque and feed rate resulted in improvements in load transfer compared with spin-to-stall installations using a 115-149 Nm nut breakout pin and 150mm/sec feed rate. Of the two changes, high bolt feed rates are judged impractical and technically inferior in most coal mine environments where, in the typical weak and laminated strata conditions, the increase in resin pressure could cause resin loss. Further, the total mixing time at the top of the drill hole due to high feed rate is less than when using the higher breakout pin-nut with the normal 150mm/sec feed rate. In addition, the test results by both observational methods and analysis of stiffness, suggest that the resin uniformity and strength was improved when using the higher nut pin breakout with a 150 mm/sec feed rate. For these reasons, the further development of the DSI Underground spin-to-stall resin has proceeded with the super high 216-298 Nm breakout pin-nut.

The results have implications for the ongoing monitoring of spin-to-stall resin bolt systems used in underground mining operations. The link between mixing time at the top of the drill hole and load transfer makes it important for operators and engineers to understand and record this measure during routine spin-to-stall bolt installations. The information can then be used to evaluate the potential load transfer in the critical upper 300-400 mm of the bolt. Should mixing time be found to be inadequate to achieve optimal load transfer then remedies can be explored to optimise bolt performance.

REFERENCES


DEVELOPMENT OF DOUBLE SHEAR TESTING OF TENDONS

Naj Aziz¹, Guanyu Yang², Saman Khaleghparast, Travis Marshall

ABSTRACT: More than 25 years of uninterrupted research on ground support technology for underground mines has been undertaken at the University of Wollongong. This research has resulted in significant findings on tendon characteristics and strength properties. The paper focuses on the development of a fourth generation of cylindrically shaped shear test apparatus for assessing tendon performance in shear. This shear apparatus is known as the MK-IV Double Shear Box or Naj Aziz Double Shear Box (NADSB), and is based on the experience gained from the development of previous versions of rectangular double shear boxes. The new NADSB is circular in shape and is fitted with a truss system, which permits friction free shear testing of tendons across joint planes. A series of double shear tests were carried out on a number of cable bolts commonly used in Australian mines, both plain and indented wires, under varied pretension loads. The results were compared with similar test results using rectangular shaped double shear apparatus, with and without friction across joint faces. The significance of wire surface roughness and increased initial pretension loads are discussed and conclusions made, suggesting that indented wires are inferior in shear compared with plain cable bolts. The general test procedure of the NADSB is described and different concrete reinforcement techniques are reported. The influence of external and internal confinement of the concrete medium blocks in circular double shear box contributed to consistent test results with a minimum of lateral and axial cracks occurring in the host medium.

INTRODUCTION

Load transfer property testing of tendons has been studied over several decades with much of the early research focused on pull and shear testing of solid rock bolts and hollow bolts (like Swellex and split set tube bolts), as these elements were used extensively at this time. It was not until the 1970’s that the use of cables for ground reinforcement developed, and as such highlighted the need to extend load transfer property testing to include shear strength of cable bolts. Initially the discarded hoisting ropes were used for ground support in metal mines, particularly in stoping wall support and then late in 1980’s the use of cable tendons started in coal mines. A summary of the historically significant studies undertaken on shear behaviour of tendons is reported in Table 1 (Jalalifar, et al., 2006).

The early methods of shear testing were carried out using direct shear testing machines under constant load conditions. Bjurstron (1974) and Ludvig (1983) used the standard constant normal load direct shear rigs to undertake general studies on the load transfer mechanism of tendons. This was further extended to cable bolt shear tests at both 45° and 90° to the sheared surface. With the exceptions of a few, these methods were mostly examined the pre-failed performance of the tendon with respect to the characteristics of the various tested parameters, such as medium strength, grout type, loading rates. Only a few tests allowed the tendon to be loaded to its final failure, and these were made with lower capacity bolts and cables which were reported by Bjurstron (1974). Others undertook studies on cable bolt shear, including Dobe (1996) and Gores, et al., (1996).

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In Australia, tendon elements are normally sheared laterally under constant normal load conditions. Notable studies were that of Dight (1982) who reported on shear testing of steel wire strands in the direct shear machine, initially developed by William (1980) but was later modified by Dight (1982) to handle cable shearing under constant normal load. The tested cable was first grouted in a plastic tube and the plastic sleeved cable was then grouted in 65 mm steel tube using cementitious based grout. Shearing was carried out perpendicular to the axis of the cable. Others undertaking shear testing in guillotine style include the works of Bigby (2005), Thomas (2012) and Aziz et al (2015).

Table 1: Chronology of tendon shear performance investigation and development since 1972 (modified from Jalalifar, 2006)

<table>
<thead>
<tr>
<th>Author</th>
<th>Base of the method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dulasccka (1972)</td>
<td>Development of plastic hinge after max. Moment</td>
<td>Prediction of shear force by bolt</td>
<td>Non static equilibrium condition in shear joint</td>
</tr>
<tr>
<td>Bjurström (1973)</td>
<td>Equilibrium forces acting on the system</td>
<td>Estimation of shear resistance: due to dowel, reinforcement and friction effect.</td>
<td>Mode of failure in surrounding materials was neglected</td>
</tr>
<tr>
<td>Hass (1976)</td>
<td>Single shear test</td>
<td>Test were performed on real rocks</td>
<td>Non-uniform stress distribution along the shear joint</td>
</tr>
<tr>
<td>Aziz (1977)</td>
<td>Single shear test</td>
<td>Different bolt angles were considered</td>
<td></td>
</tr>
<tr>
<td>Hibino (1981)</td>
<td>Single shear test</td>
<td>Pretensioning was applied</td>
<td>Pretensioning and bolt’s inclination could not considered properly</td>
</tr>
<tr>
<td>Hass (1981)</td>
<td>Single shear test</td>
<td>Real rocks with different bolt angles were considered</td>
<td>Pretensioning was not applied</td>
</tr>
<tr>
<td>Dight (1982)</td>
<td>Theoretical analysis</td>
<td>The prediction of dowel effect and hinge point was considered</td>
<td>Neglecting the bolt behaviour in elastic range, poor effect of normal stress on joint</td>
</tr>
<tr>
<td>Egger and Fernandz (1983)</td>
<td>Single shear test</td>
<td>Different bolt angles were applied</td>
<td>Pretensioning was not applied</td>
</tr>
<tr>
<td>Ludvige (1983)</td>
<td>Single shear test</td>
<td>Different bolt angles were applied</td>
<td>No fully grouted bolt was tested</td>
</tr>
<tr>
<td>Schubert (1984)</td>
<td>Equilibrium forces acting on the deformed system</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Real rocks was tested</td>
<td>Pretensioning was not considered</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yashinaka (1987)</td>
<td>Direct shear test</td>
<td>Different bolt angles was considered</td>
<td>Pretensioning could not apply</td>
</tr>
<tr>
<td>Spang and Egger (1990)</td>
<td>Single shear test</td>
<td>Real rocks was tested, max bolt contribution and displacement was predicted</td>
<td>Limited in: grout types, annulus thickness, rock strength and pretensioning</td>
</tr>
<tr>
<td>Egger and Zabuksi (1991)</td>
<td>Single shear test</td>
<td>Prediction of bolt failure at a combination of axial and shear</td>
<td>No joint confinement and bolt pretensioning was considered</td>
</tr>
<tr>
<td>Holmberge (1991)</td>
<td>The equilibrium of forces acting on the deformed bar</td>
<td>Bolt behaviour was analysed in both elastic and plastic stages</td>
<td>The effect of grout was disregarded</td>
</tr>
<tr>
<td>Ferrero (1995)</td>
<td>Single shear test</td>
<td>The plastic stage of the system was considered</td>
<td>In-capability of the method to show the effect of pretensioning</td>
</tr>
<tr>
<td>Pellet and Egger (1995)</td>
<td>Theoretical analysis</td>
<td>Both elastic and plastic stages was analysed</td>
<td>The effect of grout material was neglected</td>
</tr>
<tr>
<td>Dube, 1996</td>
<td>A laboratory study on cable bolts subjected to combined tensile and shear loads.</td>
<td>evaluating both parameters in one test</td>
<td>Operation difficulties</td>
</tr>
<tr>
<td>Goris et al. (1996)</td>
<td>Single shear test</td>
<td>Perpendicular bolts was analysed</td>
<td>Non-equilibrium load distribution on the shear joint, Max. Displacement was up to 46 mm</td>
</tr>
<tr>
<td>Grasselli (2005)</td>
<td>Double shear test</td>
<td>Symmetric situation around the shear joint</td>
<td>Bolt pretensioning was not considered</td>
</tr>
<tr>
<td>Azz et al (2005)</td>
<td>Double shear test</td>
<td>Symmetric situation around the shear joint, pretension effect, bolt profile, any grout, bolt and hole diameter</td>
<td>The size of the shear box is small for large bolt diameters and strong steel bolts</td>
</tr>
<tr>
<td>McKennie and King (2015)</td>
<td>Single shear test</td>
<td>Shearing of cable with no concrete face contact in an integrated test machine</td>
<td>Cable pretension capable of defining cable debonded.</td>
</tr>
</tbody>
</table>
CHRONOLOGY OF DOUBLE SHEAR TESTING METHODS

Rectangular Double Shear Rigs (MKI, MKII and MKIII)

The earliest reference to the use of double shear testing rigs was a paper detailing on the work carried out by the rock mechanics group of the University of Wollongong, by Aziz, et al., (2003) at the fourth Coal Operators’ Conference. The 600 mm long shear box is known as MKI Double Shear box or simply MKI DS-box, as shown in Figure 1. This box was relatively small in dimension and was found to be unsuitable for shear testing of large capacity rock bolts and cable bolts in low strength concrete. Currently DS-MKI is used for shear testing of fibre glass rods and smaller diameter mild steel rib bolts as reported by Aziz, et al. (2015 and 2016) and recently by Khaleghaparast, et al., (2019) used it for the static and dynamic tests.

To accommodate shear testing of larger capacity rock bolts and cable bolts, two new versions of double shear testing rigs were subsequently developed, as shown in Figure 2: (a) The rectangular DS-MKII box consisting of two 300 mm long outer cubic boxes and a 450 mm long middle central cuboid box with 300×300 mm² cross-sectional area. The overall length of 1050 mm has opposing concrete joint faces in contact with each other and therefore the applied shear force is spent on overcoming the combination of the shear failure load and the friction force of the sheared medium joint faces, and (b) DS-MKIII box, a modified MKII DS Box, with opposing concrete joint faces not in contact with each other and therefore the measured shear resistance force is spent only on shearing the cable strand wires, in other words, it is a frictionless shearing box. Further information on double shear test boxes MKII and MKIII was reported by Aziz, et al (2010, 2014, 2015, a and b, 2016) and Resekh, et al., (2016).

Figure 1: MKI double shear rig
Figure 2- Double shear test rigs (a) MKII and (b) MKIII.

The level of shear force spent on overcoming the friction force between joint faces was determined using the following mathematical equation model based on the combination of Mohr Coulomb criterion and Fourier series scheme (Aziz, et al., 2015a).

$$\tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^{\infty} a_n \cos\left(\frac{2n\pi}{2\pi} \cos^{-1}\left( \frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_1^2}}{24a_3} \right) \right) \right) \tan(\phi) + c$$

(1)

Where $\tau_p$ is the shear stress, $\Sigma$ is the shear load, $C$ is cohesion, $a_n$ is Fourier Coefficient, $n$ is the number of Fourier Coefficient, which varies between 0 and 3 and $T$ is the shearing vertical travel/displacement.

Aziz, et al., (2016) verified the effect of the equation with experimental test results. Excessive fractures in the rectangular shaped concrete medium contribute to the inconsistency in test results. Typical axial cracks are shown in Figure 3.

Figure 3: Cracked concrete block after shearing and zone of crushing

MK-IV CIRCULAR DOUBLE SHEAR BOX (NAJ AZIZ DOUBLE SHEAR BOX)

Figure 4 shows the general view of a new cylindrically shaped Double Shear Testing Rig (MKIV-DSB), and now called “Naj Aziz DS Box. (NADSB) The 300 mm diameter steel circular clamps permit the application of external confinement to the cylindrical concrete medium. The outer DS cylinder sides rest on support cradles to provide a stable positioning during the shearing stage, allowing the longer central part of the box to shear vertically down with a minimum of lateral movement. With the use of four steel trusses and 30 mm thick reinforced side plates, shown in Figure 4b, the arrangement permits a minimum of contacts occurring between the concrete blocks side faces during shearing, thus preventing part of shearing forces being spent on overcoming the medium joint sides rubbing friction. However, for higher axial cable pretension loads, the two 30 mm thick steel plates are further reinforced laterally with; (a) welding a 30 mm thick and 100 mm bar across top half and (b) a 20 mm thick and 200 mm square plates inserted on the cable to minimise their inward bending as shown in Figure 4 b. The whole assembly is mounted on outer base cradle half cylinders, fastened and secured to the carrier base frame using eight 20 mm diameter threaded bars. The concrete steel clamp is 17.5 mm in thickness, fastened to the lower half of the full clamp with three bolts pre side. Figure 4d shows the view of post shear test of the shear box.

The preparation of the NADSB assembling differs from the rectangular shear box. The circular concrete blocks are cast in 300 mm diameter Formatube cardboard cylinders. Two 300 mm
and one 450 mm cardboard lengths are cut and assembled in a specially prepared wood frame for concrete pour as shown in Figure 5. During casting of the concrete and production of the central hole for cable installation, a conduit wrapped with 8 mm PVC tube is held vertically along the mould to precast a rifled hole through the centre of concrete blocks. Once the concrete was poured it was left to set and harden, the steel conduit as well as the PVC tube are removed in similar fashion as reported by Aziz, et al., (2017) in ACARP project report C24012. Once a few days of casting the concrete blocks are removed from cardboards and mounted on the double shear. The lower semicircular sides steel frames of the 300 mm long side blocks are placed on steel support cradles resting on the carrier base frame. The middle section of the double shear box set-up rests temporarily on either wood blocks and lately on a purpose built semicircular table with retractable legs.

Figure 4: MKIV DSB / NADSB; a) Assembled rig with the central block resting temporarily on wood or purpose built plate; b) Axial load retaining side reinforcement; c) Assembled rig mounted on compression testing machine; d) Post-test assembled box with sheared down central block

Figure 5: Preparation of the cylindrical concrete blocks and casting of rifled holes in concrete block using PVC flexible tubes wrapped around the central steel rod

Similar to the DS-MKIII rectangular shear box set up, the NADSBuses the truss system/braces around the double shear assembly for frictionless shearing as shown in Figure 4c. The truss
system consists of four 1100 mm long 9 mm thick open channel steel braces connected between two 30 mm thick side steel plates. Next the cable bolt is inserted into the central axial hole, which is followed by mounting a suitable capacity load cell on each protruding side of the cable in the assembled concrete blocks and tensioned to a predetermined axial pretension load, using a “Blue Healer” tensioner. Tensioning of the cable is retained by the barrel and wedge retainers. When assembled, gaps of almost 5 mm are left between concrete blocks, thus the adjacent sheared concrete faces are kept apart, eliminating contact between the sheared faces. There will be no joint face surface friction force.

Once the cable is pretensioned, cementitious grout is injected from the vertically pre-cast radial holes on top of each concrete block into the hole annulus space around the cable strand. After grout curing time, the double shear assembly is placed on the carrier base frame consisting of a parallel pair of rail track sections welded on a 30 mm thick steel plate. The outer 300 mm side cube blocks of the double shear apparatus are mounted on 20 mm thick steel cradles set on 100 mm steel blocks. Next the central 450 mm long block will be freed to be vertically sheared down using a 500 t capacity hydraulic universal testing machine. The recommended rate of shearing is in the order of 1 mm/min for the maximum 100 mm vertical displacement.

The MK-IV shearing box can be modified to study the cable deboning. This is achieved by adding a long section of circular steel clamp on one side of the double shear box by dispensing with one load cell, enabling encapsulation of the longer cable bolt length. The extent of the axial load build up on the bolt during shearing process can still be monitored by the second load cell on the other side of the shear apparatus. The addition of the extra concrete column length on one side of the box may not influence the truss system performance in Joints frictionless tendon shearing.

**RESULTS AND ANALYSIS**

Table 2 lists details of several cable bolts being tested in the NADSB apparatus. All tested cables were SUMO cable bolts, plain and indented wire strands. Figure 6 shows the load displacement profiles of both types; with the peak shear load of plain strand cable being typically higher than the indented type and at greater displacement. However, the shear failure load of each wire in the strand is inconsistent compared with the plain strand.

**Table 2: Summary of double shear test results using DS MKIV rig (Naj’s DS Box)**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable</th>
<th>Test date</th>
<th>UTS (t)</th>
<th>Pt (t)</th>
<th>Peak Shear load/side (kN)</th>
<th>Peak Axial load L (kN)</th>
<th>Peak Axial load R (kN)</th>
<th>Shear displ. (mm)</th>
<th>Concrete reinforced internally with Steel tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SUMO-P</td>
<td>15-05-18</td>
<td>65</td>
<td>15</td>
<td>400</td>
<td>408</td>
<td>451</td>
<td>71</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>SUMO-P</td>
<td>25-05-18</td>
<td>65</td>
<td>15</td>
<td>377</td>
<td>275</td>
<td>294</td>
<td>43</td>
<td>yes</td>
</tr>
<tr>
<td>3</td>
<td>SUMO-P</td>
<td>06-06-18</td>
<td>65</td>
<td>2</td>
<td>451</td>
<td>422</td>
<td>442</td>
<td>86</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>SUMO-P</td>
<td>26-06-18</td>
<td>65</td>
<td>2</td>
<td>483</td>
<td>270</td>
<td>250</td>
<td>59</td>
<td>yes</td>
</tr>
<tr>
<td>5</td>
<td>ID- SUMO</td>
<td>10-07-18</td>
<td>63</td>
<td>15</td>
<td>315</td>
<td>366</td>
<td>360</td>
<td>61</td>
<td>No</td>
</tr>
<tr>
<td>6</td>
<td>ID-SUMO</td>
<td>18-07-18</td>
<td>63</td>
<td>2</td>
<td>378</td>
<td>280</td>
<td>286</td>
<td>68</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>ID-SUMO</td>
<td>02-08-18</td>
<td>63</td>
<td>15</td>
<td>240</td>
<td>174</td>
<td>173</td>
<td>44</td>
<td>yes</td>
</tr>
<tr>
<td>8</td>
<td>ID-SUMO</td>
<td>10-08-18</td>
<td>63</td>
<td>2</td>
<td>310</td>
<td>193</td>
<td>189</td>
<td>47</td>
<td>MIX</td>
</tr>
<tr>
<td>9</td>
<td>SUMO-P</td>
<td>20-09-18</td>
<td>65</td>
<td>2</td>
<td>453</td>
<td>285</td>
<td>286</td>
<td>61</td>
<td>Yes</td>
</tr>
</tbody>
</table>

P: Plain, ID: Indented cable wires, Cable dia. 28 mm. Sample 8 had one side block not reinforced
With regard to changes to test environment as listed in Table 2, the load-displacement and axial pretension build up load in plain and indented Sumo cables are shown in Figure 6. Both tests were carried out in concrete blocks externally reinforced with same steel clamps. These two cables were initially pretensioned to 15 t and are listed as test 1 and 5 in Table 2. The level of displacement profiles in both shearing process were close, particularly when defining the first peak shear load. In addition to wire indentation and in reality there are three other factors that may test results. These include the strength of the medium (concrete), the effectiveness of medium confinement, and applied axial pretension loads on bolts.

1. **Medium confinement**: Without effective confinement, internally, externally or combined, the true shearing of the tendon has been found to be difficult to assess as the host medium would end up being cracked radially and axially with the failed cable being subjected to more of tensile failure rather than shear, particularly at the hinge points in the vicinity of sheared joint planes, as shown in Figure 3.

Figure 7a shows the cable installed in concrete blocks reinforced internally with steel tube (dia: 165 mm, wall thickness: 3 mm). The internal confinement of the concrete block contribute to increased concrete strength and stiffness, which minimises early concrete deformation around the tendon close to sheared joint faces, reducing concrete deformation depth at the hinge points by as much as 50%. These result into reduced vertical cable displacement, which would be less than that occur with un-reinforced concrete medium. Figure 7b shows the load displacement profile of internally reinforced concrete of an indented cable. Clearly there were early strand wires failures prior to the strand wires total peak shear load failures. This phenomenon is reported in various indented cables irrespective of the test condition with regard to block shape and different block test environment Aziz, et al., (2016), and is the subject of further research.

2. **Cable bolt indentation**: Comparative tendon shear studies carried out using different methods of testing as reported in various published papers(Aziz, 2015, 2017, 2017 and 2018) and in ACARP Project C 24012 (Aziz, et al., 2017) revealed the following:

- Indented cables are, in general lower in ultimate tensile strength than their counter part plain wire cables. It is understood that such loss of strength may be attributed to cable wires weight loss during indentation process as reported by Aziz, et al, (2017, 2014), however, localised wires stress concentration on the wire has been found to be in higher impact than the cable wires weight loss, as reported by Aziz, et al., (2019)
• Vertical shear displacement at peak shear failure load in indented cable is less than the plain wired cable.

• Unlike plain wire failure, where the load decreases gradually with the snapping of each wire in the cable strand, the wire failure in the indented cable may not necessarily follow gradual and sequential load failures. As can be seen from Figure 7b, the initial wire snapping in the strand may not cause the ultimate peak failure load of the cable; on the contrary, there is a gradual increase in successive failure loads in the strand with greater peak failure loads. This phenomenon is currently being investigated with the view that the orientation of the initial failed wires undergoing relatively excessive bending during shearing.

3. Concrete strength and confinement: Irrespective of the test method, the concrete strength and its confinement play a significant role in tendon shearing. It will be difficult to undertake shear testing of cables, particularly stronger cables with the tensile strength (failure loads) greater than 50 t in lower concrete strength of less than 40 MPa unless it is adequately confined. Three ways can be used to increase the medium/strength;

• external confinement of the concrete with steel clamps,
• internal reinforcement of the composite medium by placing moulding steel tubes in the concrete;
• a combination of (a) and (b) techniques and shows all three types of reinforcements with 40 MPa concrete.

Reinforcement of the cylindrically shaped double shear concrete mould alone may not stop axial and radial cracking of concrete blocks. These cracks, when formed initially, may not be large but the shearing of cable will cause the crack to widen and crush, which will influence the cable behaviour in shear with increased shear displacement. The sheared cable may fail in tensile shear instead of shear. Figures 3 and 7a show two exposed reinforcements and in the first one, shown in Figure 3 the concrete confinement has an external confinement, while those shown in Figure 7a are both internal and external confinements. Without the internal confinement/ reinforcement, cracks will occur along the full length of the concrete in both rectangular as well as in circular concrete blocks, however, the extent of axial cracks in circular concrete are small with little influence on cable shear load and the width and size of the deformation or crushing zone because of the evenly distributed and effective external confinement of steel clamps. It should be noted that the internal reinforcement was also found...
to be effective in rectangular shaped medium as demonstrated in the recent study by Khaleghparast et al., 2019) in small diameter solid rock bolts.

Figure 8: changes in deformation/crushing zones at the hinge point, during shearing in different concrete mould shapes and reinforcements

COMPARING TEST RESULTS BETWEEN DS MKII, MKIII AND MKIV (NAJ AZIZ DSBOX)

Table 3 lists test results from testing Sumo cables using different shear boxes of MKII, MKIII and MKIV. Both Sumo plain and indented cable were tested in externally confined 40 MPa concrete with pretension loads as indicated in the table. Testing with MKII meant the existence of contact joint faces friction, in which part of applied shear load was spent in overcoming friction. Testing using MKIII and MKIV boxes required no joint face friction force. The following points are noted from Table 3 and load-displacement figures reported by Li, et al (2017)

- The external steel confinement generally strengthens the integrity of the host medium during shearing, however circular clamps or confinements is more effective method than the rectangular type because of uniform lateral confining loads applied all around the concrete.
• The shear failure loads tested in the circular NADSB (MKIVDSB) are generally lower in comparison with test results from both MKII and MKIII boxes. This is because all-round confinement load is uniform and strengthens concrete stiffness. The higher shear load values, particularly when testing with MKII box is attributed to the additional shearing force needed to overcome the contact frictional forces.

• The shear load values determined when using the NADSB are relatively closer to Megabolt Single Shear Test results (MSST) as reported by Aziz et al in ACARP report C24012(2017). These values occur in testing cables that are of indented types and do not debond.

• Excessive crushing and deformation of the concrete at and near cable hinge points may lead to increased shear displacement resulting in to greater shear force values indicating that the sheared section of the cable wires are in near tensile failure rather than in shear, as shown in Figure 8. This may have some benefit for ground reinforcement particularly in softer formation with excessive lamination and shearing.

Table 3: Test data on Sumo plain and indented cables tested in MKII, MKIII and MKIV double shear boxes(NADSB), all externally clamped. Cable diameter- 28 mm.

<table>
<thead>
<tr>
<th>Apparatus type</th>
<th>Cable type</th>
<th>UTS (t)</th>
<th>Pretension (t)</th>
<th>Peak shear load per side (KN)</th>
<th>Shear load at 70% peak shear load (KN)</th>
<th>Peak axial load (L (KN))</th>
<th>Peak axial load (R) (KN)</th>
<th>Shear displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MKII</td>
<td>Plain SUMO</td>
<td>65</td>
<td>10</td>
<td>1318±7.659</td>
<td>461</td>
<td>365</td>
<td>418</td>
<td>78.99</td>
</tr>
<tr>
<td></td>
<td>ID SUMO</td>
<td>65</td>
<td>25</td>
<td>1422±7.111</td>
<td>408</td>
<td>342</td>
<td>417</td>
<td>58.76</td>
</tr>
<tr>
<td>MKIII</td>
<td>Plain SUMO</td>
<td>65</td>
<td>0</td>
<td>896±2.443</td>
<td>342</td>
<td>260</td>
<td>298</td>
<td>32.8</td>
</tr>
<tr>
<td></td>
<td>ID SUMO</td>
<td>65</td>
<td>15</td>
<td>852±2.426</td>
<td>364</td>
<td>433</td>
<td>433</td>
<td>88.2</td>
</tr>
<tr>
<td>MKIV (NADSB)</td>
<td>Plain SUMO</td>
<td>65</td>
<td>15</td>
<td>767±2.383</td>
<td>376</td>
<td>388</td>
<td>85.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ID SUMO</td>
<td>65</td>
<td>15</td>
<td>767±2.383</td>
<td>376</td>
<td>388</td>
<td>85.7</td>
<td></td>
</tr>
</tbody>
</table>

* No cable failure

CONCLUSIONS

MKIVDSB (Naj Aziz DSB) method is an effective and reliable system for determining the shear strength of tendons. The circular shape of the steel clamp allows a uniform application of confinement around the concrete perimeter contributing to reduced depth of deformation or cracking zone, enabling the tested tendon to fail in quasi shear rather than in tension.

The cylindrically shaped moulds can be effectively confined internally thus reducing the formation of wider axial cracks that would influence tests conditions. The internally reinforced concrete medium allows shear testing of cable in stronger strength confinement. Internal reinforcement of the concrete medium was found also to be effective in rectangular shaped medium, thus rendering the concrete shape factor as irrelevant.

The test findings from the NADSB study agreed with single shear testing results undertaken by Aziz, et al., (2017) in the ACARP C 24012 report.

The NADSB/MKIVDSB can be modified by dispensing with one load cell and replacing it with a long section of circular steel clamp, enabling encapsulation of the longer cable bolt length for cable bolt debonding studies. The extent of the axial load in the bolt during the shearing process is still monitored by the second load cell on the other side of the shear apparatus.
REFERENCES


Dobe, S, 1996. A laboratory study on the behaviour of cable bolts subjected to combined tensile and shear loads, MSC thesis, Queen's University, Kingston, Ontario, Canada.


Thomas, R, 2012. The load transfer properties of post-groutable cable bolts used in the Australian Coal industry, In proc 31st International Conference on Ground Control in Mining, Morgantown, USA, pp.1-10 (Edt: SS Peng)


WHY THE PEAK SHEAR LOAD OF INDENTED CABLES INCREASES WITH INCREASED WIRE FAILURES?

Guanyu Yang¹, Saman Khaleghparast, Naj Aziz², Jan Nemcik, Travis Marshall

ABSTRACT: In shear testing of indented cables it has been found that indented cables peak share load failures behave contrary to the normal failure behaviour. The gradual strength loss with each individual wire failure in an indented cable strand may not lead to subsequent peak shear failure of the remaining strands in decline. This failure behaviour is characteristic of indented cables and occurs irrespective of the test method used (single shear or double shear test). Accordingly in this study all wires in a tested cable strand were instrumented with strain gauges. Each instrumented wire was individually colour coded to assist in determining the location of the wires in the strand circumference with respect to the direction of shear. The location of wires in the perimeter was identified at the sheared joint surface areas. During testing of the cable using a circular MKIV double shear apparatus (Naj Aziz DS Box) the initialisation of wire failure was identified by the strain gauge readings. This data found that the wires failing early were located on the upper segment of the bent strand during shearing process, indicating that the indentations introduced stress concentration spots on the wire, causing the strand wires to fail prematurely with less tolerance to bending than smooth wired cable.

INTRODUCTION

The use of long tendon ground support elements (cable bolts) is now common practice in modern underground coal mines, hard rock mines, tunnels and other underground structures. Due to their material properties cable bolts contribute significantly to the overall ground reinforcement provided by a support system. Cable properties in tension and shear are in many cases vital to maintaining a safe and productive underground environment. Accordingly, tension and shear properties must be assessed accurately using both valid and reliable methods.

Testing of tendons for tension is a common method of evaluating the load transfer properties of tendons and also for strength. This is reported by various researchers and can be undertaken both in the field and in the laboratory (Stillburg, 1984, Fuller 1983, Windsor (1992) Hagan and Chen, 2015, Hyett, et al., 1992., Aziz, et al., 2014, Tadolini, et al., 2012, Thomas, 2012). Tendon shear testing methods can differ with varied purpose and outcome. Until recently cable strength and load transfer capacity of cables were examined using a guillotine type single shear testing apparatus, which is, based on the British Standard (BS7861- Part 2, 2009). Recently there has been the increasing interest on shear testing of cables with the focus being directed to cable or tendon load transfer capacity as well as cable de bonding, this emphasis has led to the development of more credible methods and currently the testing of tendons is more closely simulated to ground conditions in which cable shear testing is carried out in concrete of varied strength and using double shear methods as reported by Aziz et al., 2010, 2014, 2015, and 2016, and single shear method by McKenzie and King (2015) and Aziz, et al., (2017, 2018) and Yang, et al., (2018). The later tests with simulated conditions demonstrated that the profile and surface conditions of the strand wires affect their load transfer characteristics. Some cable

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wires’ failure during testing, particularly by shear testing, show that some strand wires fail less than the final peak shear load. In particular the indented cables appear to contribute to such unusual wires failure behaviour. Clearly, there is a need to shed light on this failure phenomenon and determine what contributes to some wires’ premature failure. This is the subject of study in this paper.

SINGLE AND DOUBLE SHEAR APPARATUS

Shear testing of tendon rock bolts and cable bolts can be carried out using either the single shear or double shear testing apparatus.

Single shear apparatus

The conventional guillotine type apparatus based on British Stand BS 8761-part 2 (2009) and Megabolt single shear apparatus are methods for shear testing of tendons. The Megabolt single shear apparatus (McKenzie and King, 2015) is an integrated system that incorporated 120 t compression testing machine to the system and is also capable of evaluating bonding and debonding characteristics of tendon with respect to surface roughness. The performance of both types of rigs for testing cable bolts, as shown in Figure 1, are also reported by Aziz, et al in (2015 and 2018 a, b) and Mirzaghorbanali, et al., 2017 respectively.

Figure 1: Single shear apparatus; (a) British standard based single shear and (b) Megabolt single shear apparatus

Double Shear (DS) apparatus

Four types of double shear apparatus have been developed over the years by researchers at the University of Wollongong. Three types are rectangular box types and the fourth circular shaped. The four are listed as:

- MKI- 150 mm x 150 mm x 300mm.rectangular box
- MKII - 300 mm x 450 mm x 300 mm rectangular box. A larger box, double the size of MKI box
- MKIII - 300 mm x 450 mm x 300 mm rectangular box. This is the same MKII rig but fitted with lateral truss system to eliminate friction between the joint faces. In this situation the applied shearing force will be totally spent of shearing the cable.
- MKIV – 300 mm x 450 mm x 300 mm circular shaped box, which is also as known as Naj Aziz DS Box.

The above mentioned double shear box types, with and without lateral truss system are reported by Aziz, et al., (2019)

Cable strand wires performance in shear
Several cables tested in shear using various types of the UOW double shear boxes have resulted in different failure behaviours of strand wires. The variance in failure pattern is related to the surface condition of smooth or indented wires as shown in load – displacement in Figure 2. The peak failure load of a cable bolt would normally be the maximum load and occur as the first wire failed with each subsequent wire failure continuing to shed load. Some indented wires appear to fail in an abnormal fashion, particularly when compared with smooth wire type, in which its peak failure load is lower than the smooth wired cables and may not be reached when the first wire fails. The subsequent wire failures may lead to increased peak shear load with the failure of indented wires. No abnormal failures were found in plain/ smooth wire cables with the first wire failure always being the peak load. Abnormal load failures in indented wire cables may also lead lower ultimate peak load failures.

**Figure 2: Shear testing of (a) plain and (b) indented cable bolts in 40 MPa concrete subjected to 10 t pretension load**

**WHY ABNORMAL FAILURES:**

Based on several shear tests, it was observed that, due to the location of different wires along the perimeter of the cable with respect to the direction of shearing force, some indented wires were subjected to extra bending and early snapping in comparison with others in the cable periphery. The wires in question are those located on the top half of the cable perimeter with respect to the direction of shearing. Three possible factors may contribute to the abnormal failure of indented wires in the cable strand; (a) excessive bending of wires residing on the top part of the cable strand with respect to the wire location on the perimeter and shear direction and (b) wire weight loss due to indentation. This loss may cause up to 10% decrease in peak load depending on tested cable tyre as reported by Aziz, et al., (2015), and (c) development of localised stresses due to the indentation process of the wires.

**TEST PROCEDURES**

**Pull testing of wires under varying stress conditions**

To demonstrate the strength performance behaviour under different test environments, three 400 mm long sections of the same cable bolt wire were cut and subjected to various forms of stress environments, as described above. Figure 3a shows three wires with one wire subjected to impact punch, the second was spot welded and the third one was intact. All three wires were then pull tested to failure using 50 t capacity Instron universal testing machine. The tested wires are shown in Figure 3b. The peak failure load of the intact wire was 69 kN, the failure load of punched wire was 67 kN and spot welded wire was 51 kN. All tested wires were from Sumo cable strand. This finding demonstrated clearly that failure of wires varies with respect to test wire conditions and that excessively stressed wire fails prematurely.
Monitoring wire strength by instrumentation shearing of the cable bolt

To obtain reliable results on the location of the early wire failure and their identification, all wires of the cable strand periphery were painted with different colours, using oil-based paints as shown in Figure 4a. The orientation of each wire in the sheared joint areas was clearly marked. Then one strain gauge was installed on each strand wire and the wires were numbered and marked for identification in relation to wire location in the perimeter of the strand and the shear force direction. The application of the strain gauges required a clean and flat surface on each wire, which was achieved by sanding a small area of the wire surface, located some distance away from the sheared section so that the glued strain gauge will not be affected by changes in the wire cross section area. The area was polished and wiped clean with an alcohol based cleaner to ensure any impurities on the wire surface were removed. All strain gauges were subsequently checked for functioning and line continuity prior to start of shear loading. The profile of the cable wires, location at both sheared joint faces were identified and drawn as shown in Figure 4b. The strain gauge wires were carefully coursed out of the circular MKIV double shear box in such a way that all wires were not damaged during assembling and subsequent shearing. Figure 5 shows the assembled and instrumented Naj Aziz double shear box mounted on a 500 t compression testing machine.
RESULTS AND DISCUSSIONS

Figure 6 shows the load-displacement graph of the instrumented cable under shear and Figure 7 shows the individual strain gauge readings from different wires. The first wire (R9) in the strand snapped at 25.6 mm of displacement, at shear load of 338.4 kN and occurred on the right side (R side) of the double shear testing joint face area. This was followed by the failure of the second wire (R8) at 27 mm displacement. However, the maximum shear load of 443.1 kN of R7 was reached before failure of R7 wire occurred at 38.2 mm displacement. This is followed by erratic loads failures, occurring in indented wire strand and is contrary to the past test results from plain cables as reported by Aziz, et al., (2015, 2016, and 2017). In plain wire strand the peak load decreases gradually with each subsequent wire failure, as shown in Figure 2a.

During the early stage of shearing process some cable strand wires, namely Green (R2), Red (R3), Blue (R4), Black (R5), orange (R7), were subjected to early negative strain of shear load-
displacements as shown in Figure 7, while others Yellow (R1), Dark Blue (R6), Metallic (R8) and White (R9) recorded positive strains right from the start of shearing. This suggests that those wires with positive strain were subjected to tensile and shear failure and they were mostly located on the top side (upper side) of the strand with respect to the vertical shear direction. The failed wires are characteristically either in tensile shear or in tension with cone and cup as shown in Figure 8. These failure patterns are documented in Table 1.

![Figure 7: Strain vs displacement of instrumented wires during shearing](image)

**Table 1: pattern of wires failure on the RHS Joint shear side**

<table>
<thead>
<tr>
<th>Wire</th>
<th>Colour as seen on the RHS joint face</th>
<th>Location</th>
<th>Observed Failure pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>Yellow</td>
<td>Topside</td>
<td>Tension</td>
</tr>
<tr>
<td>R2</td>
<td>Green</td>
<td>Topside</td>
<td>Tension</td>
</tr>
<tr>
<td>R3</td>
<td>Red</td>
<td>Topside</td>
<td>Tension</td>
</tr>
<tr>
<td>R4</td>
<td>Blue</td>
<td>Topside</td>
<td>Shear</td>
</tr>
<tr>
<td>R5</td>
<td>Dark red (Maroon)</td>
<td>Bottom</td>
<td>Tensile/shear</td>
</tr>
<tr>
<td>R6</td>
<td>Dark Blue (Navy)</td>
<td>Bottom</td>
<td>Tensile/shear</td>
</tr>
<tr>
<td>R7</td>
<td>Orange</td>
<td>Bottom</td>
<td>Tension</td>
</tr>
<tr>
<td>R8</td>
<td>Metallic</td>
<td>Bottom</td>
<td>Tensile/shear</td>
</tr>
<tr>
<td>R9</td>
<td>White</td>
<td>Bottom</td>
<td>Tensile/shear</td>
</tr>
</tbody>
</table>

RHS- Sheared Cross section
CONCLUSIONS

- Cable wire failure under shear across the joint plain all occur as wires undergo excessive bending and stretching as expected. Wires located on the topside of the cable would fail in combination of pure tension and tensile shear, while wire on the opposite side exhibit only tensile shear failure. The early wire failures appear to occur on wires that are closest to the direction of the applied shear load.

- Cable strand wire Indentation may not be an advantage to cable strength and shear performance. Indentation process introduces stress zones, which when loaded in the double shear apparatus lead to earlier wire failure (lower load and lower displacement compared with smooth wire), with failure initiation at the indentation. This appears to be due to that area being subjected to a local stress.

- Wire weight loss due to indentation process, may contribute to early load failures, but it is not deemed a significant factor. Depending of the type of indentation made on the wire, the strength loss of the indented wire typically varies between 2 -10 %.

- There were irregularities in the order of peak shear load failures. Contrary to general belief that each wire snapping would lead to a subsequent reduction in peak load of the strand. No such case was found when tested in shear. Further tests are required as it is important to validate the true performance of strand wire indentation for effective application in varied strata formations and with reference to the benefits of cable debonding.
ACKNOWLEDGMENTS

Jennmar Australia supplied bolts and Minova supplied grouts for this programme of research. The new MKIV circular double shear rig, now known also Naj Aziz DS Box was Auto Cad drawn by Richard Gasser, the technical staff, Faculty of Engineering and information Sciences, UOW.

REFERENCES


Dobe, S, 1996. A laboratory study on the behaviour of cable bolts subjected to combined tensile and shear loads, MSC thesis, Queen’s University, Kingston, Ontario, Canada


FIELD AND SIMULATION STUDY FOR ROCK BOLT LOADING CHARACTERISTICS UNDER HIGH STRESS CONDITIONS

Petr Waclawik¹, Sahendra Ram², Ashok Kumar³, Radovan Kukutsch⁴, Adam Mirek⁵ and Jan Nemcik⁶

ABSTRACT: Rhomboid shaped coal pillars (35 m x 30 m to 26 m x 16 m) were formed by a modified Room and Pillar method below 850 m depth from surface at the CSM mine in the Czech Republic. The pillars were developed in a shaft protective pillar by driving roadways of 3.5-4.5 m in height and 5.2 m in width within Panel V of Seam No. 30. Development of pillars at such great depth is prone to spalling/fracturing (pillar rib dilation) due to redistribution of the high stress regime. The induced stress driven dilation was measured during partial extraction of the coal seam within the shaft protective pillar using rib extensometers. In order to stabilize the pillar ribs, four rows of rock bolts with 2.4 m length were installed into the pillar from all sides at different heights. The immediate roof was also supported by rock bolts at a 1 m grid pattern. Three-way intersections were made to control the deformation of developed pillars and other underground structures. Further, an attempt was made to understand the rock bolt loading characteristics at different stages of rib dilation using numerical modelling with the available properties of rock mass and reinforcement for the studied site. Elastic and Mohr Coulomb strain-softening constitutive models are considered in FLAC3D to evaluate the performance of the rock bolts. Results obtained on numerical models were found to be in good tune with the rock bolt loading characteristics monitored during the field study. This paper presents a discussion about the impact of rib bolting on pillar safety factor and induced load on rock bolt with respect to the dilation/spalling of pillar ribs at the studied site.

INTRODUCTION

To extract blocked coal reserves from the CSM mine of the Czech Republic, locking high grade of coal in shaft protective pillar at great depth (850 m) was partially extracted by a modified Room and Pillar method (only development). The aim of developing the protective pillar was to avoid the incidences of any surface and sub-surface subsidence in order to maintain intactness of the shaft (Waclawik et al., 2018). The mine is situated in the Karvina sub-basin of the Upper Silesian Coal Basin of the country and having a complex geological structures (Waclawik et al. 2013, Grygar and Waclawik 2011). Further, at great depth consisting of higher strength of rock mass has led to high stress conditions over the working seam. Near the trialled area, the ratio of major and minor in situ horizontal stresses to vertical stress ranged from 0.6 to 2 (Waclawik et al. 2017). However, in situ stress measured by CCBO stress overcoring cells (Obara and Sugawara, 2003; Stas, Knejzlik and Rambousky, 2004; Waclawik et al. 2016) found to be low due to influence of present roadways excavation during measurements. The ratios of the major and minor in situ horizontal stresses to vertical stress (calculated from these overcoring measurements) were 1.3 and 0.64 respectively. Thickness of coal seam No. 30 is around 4 m and contains an inter-burden parting of 0.5 m of siltstone. Average uniaxial
compressive strength of coal, roof and floor strata is 14 MPa, 105 MPa and 60 MPa respectively (Waclawik et al. 2017). First panel V was developed in seam No. 30 on a trial basis. The development was done panel-wise by keeping only two pillars along the width. The pillars are developed considering the Mark-Bieniawski strength formula and stability factor (Das, 2012; Mark and Chase 1997). The immediate roof and pillar rib from all sides were supported by resin grouted rock bolts of 2.4 m with wire mesh. Load induced over instrumented rock bolts in immediate roof was also measured during the different stages of development. Considering the geo-mining condition of the Panel V, numerical modelling was done to understand the loading characteristics of the rock bolts using FLAC3D tool. Elastic and Mohr Coulomb strain-softening (MCSS) constitutive models were considered to evaluate the performance of the rock bolts inside the rib and immediate roof. The result of numerical modelling revealed that the induced load over roof bolts installed in the roof is relatively less than rib bolts which were found to be closer to the values of field observations.

FIELD STUDY

Field study was conducted during development of Panel V of the CSM mine. The roadways were developed by Bolter Miner with 1.5-2.5 m cut-out distance followed by installation of rock bolts in roof and pillars rib. Rock bolts were used as a primary means of support in the development working considering standard design methods in the Ostrava–Karvina coal basin (OKD a.s, 2012). Rock bolts in the roof were installed at 1 m grid pattern. However, four rows of rock bolts were installed into the pillar from all sides at different heights keeping a 1 m interval between columns of bolts. Further, the immediate roof was also additionally supported by the two bolts in a row at a 1 m grid pattern up to 25 m distance from the working face. Intersections were also supported by flexi bolts. A number of geo-technical instruments including laser scanning equipment in roadways were used in this panel to monitored strata behaviour (Waclawik et al. 2017). Two developed pillars in the Panel V were monitored continuously for around 40 months. Considering the performance evaluation of rock bolts, this paper is discussing observation of roof/rib extensometers and strain-gauged based instrumented rock bolt as shown in part plan of Panel V (Figure 1).

![Figure 1: Part plan of Panel V showing locations of geo-technical instruments with orientation of major (SH) and minor (Sh) horizontal stress components](image-url)
Roof displacement was observed at different locations (Figure 1) by multipoint extensometers at five different roof horizons (1-8 m) from the ceiling of the roadways. No significant roof displacement was observed in roadways of the Panel V. Maximum 7.8 mm roof displacement was observed at an intersection. Except for a few locations, which were influenced by geological disturbances, immediate roof was found to be quite intact (Figure 2). Floor heaving was not measured by geo-technical instruments due to machine operational constraints, but from 3D laser scanning was interpreted as more than 1 m. Due to efficient rib rock bolts, pillar dilation was, relatively, less in the region between 0-1.5 m. However, significant pillar dilation was observed at the depth of 1.5-5 m. A range of 212-300 mm dilation was observed in Pillar V2 of Panel V. Induced load over nine pairs of strain gauged based instrumented rock bolt installed in the immediate roof strata (Figure 3) was observed. A range of 1-6.8 ton induced axial load over instrumented rock bolts was observed. The instrumented rock bolt could not be installed in pillar rib due to poor strength of coal under the high stress conditions.

**Figure 2: Field observation of roadways in Panel V**

<table>
<thead>
<tr>
<th>Gauge position (m)</th>
<th>0.2</th>
<th>0.45</th>
<th>0.75</th>
<th>0.95</th>
<th>1.2</th>
<th>1.45</th>
<th>1.75</th>
<th>1.95</th>
<th>2.2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td></td>
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</table>

**Figure 3: Instrumented rock bolts with nine pair strain gauges, placed along length of the bolt**

**NUMERICAL MODELLING**

Attempted field measurement for the performance of rock bolting provided valuable information, but a systematic parametric study was also needed to support the findings during the field studies. Numerical modelling is a popular tool (Ram et al. 2017; Singh et al., 2016; Murali Mohan et al., 2001) to design mining structures under varying geo-mining conditions. Numerical modelling provides an idealised laboratory condition to study the influence of different geotechnical parameters over stability of an underground structure. An investigation was done using FLAC3D package, which adopts finite difference method. In this study, an elastic constitutive model was used for strongest strata. In elastic model, loading characteristics of rock bolting in the numerical model is found to be closer to their actual field behaviour (Basarir et al. 2015) due to development of maximum elastic stress. The field observation revealed that the immediate roof strata hardly deformed due to its higher strength value. However, the immediate floor started deteriorated with time. Under such circumstances, the immediate floor
including coal seam and other strata were given MCSS and elastic properties respectively in the numerical model. The elastic model of FLAC 3D, incorporating the Sheor ey failure criterion (1997) for the rock mass was used for this study. Basically, this criterion uses the 1976 version of rock mass rating (RMR) of Bieniawski (1976) for reducing the laboratory strength parameters to give the corresponding rock mass values. This criterion is defined as:

\[
\begin{align*}
\sigma_1 &= \sigma_{cm}(1 + \frac{\sigma_3}{\sigma_{cm}})^{b_m} \quad \text{MPa} \\
\sigma_{cm} &= \sigma_c \exp\left(\frac{RMR-100}{20}\right) \quad \text{MPa} \\
\sigma_{tm} &= \sigma_t \exp\left(\frac{RMR-100}{27}\right) \quad \text{MPa} \\
b_m &= \frac{b^{RMR/100}}{100} \quad b_m<0.95
\end{align*}
\]

where, \(\sigma_1\) is tri-axial strength of rock mass or major principal stress in MPa, \(\sigma_3\) is confining stress or minor principal stresses in MPa, \(\sigma_c\) is compressive strength of intact rock in MPa, \(\sigma_t\) is tensile strength of intact rock in MPa, \(b\) is exponent of intact rock (0.5), which controls the curvature of tri-axial curve, \(\sigma_{cm}\) is compressive strength of rock mass in MPa. In the above equations, the subscript \(m\) stands for the rock mass.

The factor of safety is defined as:

\[
SF = \frac{\sigma_1 - \sigma_{3i}}{\sigma_1 - \sigma_{3i}} \quad \text{when } -\sigma_{3i} > \sigma_{tm}
\]

Otherwise,

\[
SF = \frac{\sigma_{tm}}{-\sigma_{3i}}
\]

where, \(\sigma_{3i}\) is induced major principle stress in MPa, \(\sigma_{3i}\) is induced minor principle stress in MPa.

Prior to simulation of rock bolt loading characteristics, the model is validated with pillar design strength formula used in this mine. Stability factor of pillars in the panel of the CSM mine was estimated using Mark-Bieniawski pillar strength formula (Mark and Chase 1997). Further, a recent study (Kumar et al. 2018) was carried out for performance of pillar at great depth considering Sheorey pillar strength formula (Sheorey, 1992), which incorporated the influence of depth on pillar strength was also validated here. The properties of rock mass used for the modelling were determined in the laboratory using core samples collected from the field (Table 1). These properties were used to simulate single rhomboid pillars and attempted to validate with Mark-Bieniawski and Sheorey pillar strength formulae (Figure 4). As per the Mark-Bieniawski and Sheorey formulae, strength of the pillar is 35.32 MPa and 34 MPa respectively. Results (32 MPa) obtained from the simulated study found to be close to these empirical pillar strength formulae. The same rock mass properties are used for simulation of Panel V.

<table>
<thead>
<tr>
<th>Strata</th>
<th>E</th>
<th>v</th>
<th>K</th>
<th>G</th>
<th>d</th>
<th>(\sigma_c)</th>
<th>(\sigma_t)</th>
<th>RMR</th>
<th>(\sigma_{cm})</th>
<th>(\sigma_{tm})</th>
<th>(\Phi)</th>
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<tr>
<td>Roof siltstone</td>
<td>23</td>
<td>0.14</td>
<td>10.65</td>
<td>10.09</td>
<td>2650</td>
<td>128</td>
<td>12.8</td>
<td>60</td>
<td>17.323</td>
<td>2.909</td>
<td>38.36</td>
</tr>
<tr>
<td>Roof sandstone</td>
<td>35</td>
<td>0.15</td>
<td>16.67</td>
<td>15.22</td>
<td>2690</td>
<td>155</td>
<td>15.5</td>
<td>65</td>
<td>26.935</td>
<td>4.24</td>
<td>39.57</td>
</tr>
<tr>
<td>Floor siltstone</td>
<td>18</td>
<td>0.18</td>
<td>9.38</td>
<td>7.63</td>
<td>2600</td>
<td>60</td>
<td>6</td>
<td>50</td>
<td>4.925</td>
<td>0.942</td>
<td>35.90</td>
</tr>
<tr>
<td>Floor sandstone</td>
<td>35</td>
<td>0.15</td>
<td>16.67</td>
<td>15.22</td>
<td>2690</td>
<td>155</td>
<td>15.5</td>
<td>60</td>
<td>20.977</td>
<td>3.523</td>
<td>38.38</td>
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<tr>
<td>Coal</td>
<td>2.6</td>
<td>0.25</td>
<td>1.73</td>
<td>1.04</td>
<td>1400</td>
<td>14</td>
<td>1.4</td>
<td>40</td>
<td>0.697</td>
<td>0.152</td>
<td>33.36</td>
</tr>
</tbody>
</table>

E= Young modulus in GPa, v= Poission ratio, K= Bulk modulus in GPa, G= Shear modulus in GPa, d= density of rock mass in kg/m³, \(\sigma_c\)= Uniaxial compressive strength of intact rock in MPa, \(\sigma_t\)= Tensile strength of intact in MPa, RMR= Rock Mass Rating, \(\sigma_{cm}\)= Uniaxial strength compressive of rock mass in MPa, \(\sigma_{tm}\)= tensiles trength of rock mass in MPa and \(\Phi\)= Angle of internal friction in degree.
Considering the site conditions of the studied Panel V, a model 195 m long, 113 m wide and 104 m high was developed (Figure 5) for the study. In this model, theoretical value of vertical in situ stress (0.025 x depth of cover, MPa/m) was applied. As per the actual measured horizontal in situ stress near the working panel, major and minor horizontal to vertical in situ stress set to ratio of 1.3 and 0.64 respectively in the simulated panel. Directions of the major and minor horizontal stresses are applied along with the width and length of the panel. The calibrated MCSS and elastic properties used for simulating one pillar were directly used in the developed numerical model. The size of pillars was kept as per actual development in this panel. Width and height of gallery was fixed at 5 m and 4 m respectively. A truncated load of 20 MPa (0.025 x depth of cover) for the unmodelled portion of the overlying strata was applied over the model. The sides and bottom boundaries of the model were fixed and the top was kept free. Development of the coal seam in the simulated model has been carried out till the strata monitoring period. Rock bolts of 2.4 m in roof and in pillars were installed (Figure 6) in the model as per actual practice in field.

Properties of different elements of reinforcement and other parameters reported by Holy (2018) were used in CSM mine were considered during the simulation. Grout stiffness, $K_g$, and cohesive strength, $C_g$, are determined (FLAC3D, 2012) using equations 7 and 8 respectively.

$$K_g = \frac{2\pi G}{10 \ln(1 + \frac{t}{d})}$$

$$C_g = \pi (D + 2t) \tau_{peak}$$

where $G$ is grout shear modulus, $t$ is annulus thickness, $D$ is diameter of roof bolt, $\tau_{peak}$ is shear strength of grout/rock or bolt/grout interface. Considered properties of reinforcement materials are given in Table 2.
Figure 5: Block formation for the considered site in FLAC3D

Figure 6: Development of coal seam in panel V

Table 2: Properties of reinforcement

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout</td>
<td>Grout stiffness per unit length (N/m²)</td>
<td>$1.7\times10^{10}$</td>
</tr>
<tr>
<td>Grout</td>
<td>Grout cohesive strength (N/m)</td>
<td>$2.6\times10^5$</td>
</tr>
<tr>
<td>Grout</td>
<td>Grout exposed perimeter (m)</td>
<td>$8.95\times10^{-2}$</td>
</tr>
<tr>
<td>Grout</td>
<td>Shear strength of grout/rock or bolt/grout interface (MPa)</td>
<td>3</td>
</tr>
<tr>
<td>Rock bolt</td>
<td>Cross sectional area (m²)</td>
<td>$3.8\times10^4$</td>
</tr>
<tr>
<td>Rock bolt</td>
<td>Young’s modulus (GPa)</td>
<td>200</td>
</tr>
<tr>
<td>Rock bolt</td>
<td>Tensile Yield strength (N)</td>
<td>$2.43\times10^5$</td>
</tr>
<tr>
<td>Rock bolt</td>
<td>Pretension (N)</td>
<td>$2.94\times10^4$</td>
</tr>
</tbody>
</table>
IMPORTANT FINDINGS

Under the existing high stress conditions and strong overlying strata, the observed value of pillar dilation (up to 300 mm) was more than the displacement of immediate roof (7.8 mm). Further, development of axial load on rock bolts depends on the bonding strength between the bolt/rock-grout interfaces and deformation within rock bolted zone also. In Panel V, axial load over instrumented bolts was measured at five different locations including three-way intersections. A range of 1-6.8 tons axial load developed over instrumented bolts which was around 70% less than the bearing capacity of the installed rock bolts. Maximum roof displacement and pillar dilations were observed to be 10.36 mm and 292 mm respectively (Figures 7 and 8). In the simulated model, higher axial load over rock bolts observed in rib bolts than roof rock bolts, which might be due to the influence of displacement within rock bolted zone. In the simulated mode, maximum values of tensile axial load were found to be in the middle of the rock bolts. Generally, average rock load over roof rock bolts was observed to be less than 10 tons, at some location it was observed up to 22 tons. Generally, more axial load observed in rib rock bolts, compared to the roof rock bolt. Maximum observed values of the axial load over rib bolts found to be 27 tonnes. It was found that rib bolts was installed parallel to the direction of major horizontal stress, received relatively higher value of axial load than those installed along minor horizontal stress direction (Figure 9). It might be due to the release of higher strain energy in the major horizontal stress direction compared to the minor direction of horizontal stress with rib dilation due to discontinuity of the coal seam by roadways. However, in order to establish such unique observation, extensive modelling is required to be simulated. Axial loads observed over rock bolts at five different locations are given in Figure 10.

Figure 7: Vertical roof displacement in roadways

Figure 8: Pillar dilation observed in simulation of panel V
CONCLUSIONS

Loading characteristics over rock bolts under higher stress conditions were examined using numerical models. Less roof displacement was observed in roof compared to pillar dilation, which might have occurred due to the presence of competent roof having higher uniaxial compressive strength. The numerical modelling revealed that more axial load was observed in rib rock bolts than roof rock bolts. The simulation was done by considering only four roof rock bolts in a row (excluding additional supports) and an average 10 tonnes axial load was observed, which seems to be competent enough to support the existing site conditions of the panel. In the simulated model, the maximum 27 tonnes load observed over rib rock bolts with 60-292 mm pillar dilation. This is a preliminary study on numerical modelling based on performance of rock bolts at higher stress conditions. Extensive and intensive studies are required by replicating the actual site conditions in order to establish the effect of major and minor horizontal stress.
REFERENCES


Das T, 2012. Pillar design for room and pillar mining method within the shaft pillars of CSM mine, A draft report.


APPLICATION OF SMARTEHAR TO DEFINE ROCK MASS SHEAR ABOUT COAL MINE ROADWAYS

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ABSTRACT: Shear movement of strata surrounding excavations is a critical factor in overall excavation stability. Understanding the location, timing, magnitude and direction of shear failure surrounding an excavation is useful for assessing primary and secondary roadway support requirements. A new routine shear monitoring instrument known as the SmartShear has been developed to accurately measure shear movement at multiple locations within a borehole. The shear movement is measured on a two dimensional plane at 90 degrees to the installation of the instrument using a series of tiltmeter sensors. Continual change in shear direction and magnitude over time is measured once the shear locations are detected.

This paper presents the findings from a field trial at Oaky North Mine, using the SmartShear system, assessing shear movement surrounding coal mine roadways. The timing and magnitude of shear movement along bedding planes and geological contacts has been measured during drivage of roadways. This paper is part of the ACARP project C25060.

INTRODUCTION

The successful introduction of routine roof deformation monitoring in the mid-late 1990’s using Tell Tale based systems (MacGregor 1998), is now widespread in the Australian underground coal industry. The ability to routinely monitor ground behaviour about coal mine roadways enables the management of strata control hazards through the timely and systematic implementation of controls through the application of TARP based processes.

It is recognised that some modes of ground response are associated with little or no dilation of the rock mass. This can occur along discrete geological boundaries (shearing along claybands) and/or along localised failure surfaces through the rockmass.

The SmartShear is a cost effective, routine shear monitoring instrument that can easily be installed in open boreholes via a spring loaded mechanical anchoring system. The SmartShear is made up of a series of Micro Electrical Mechanical Systems (MEMS) tiltmeters that can be installed up to 10 m into open boreholes. Providing sufficient resolution to resolve discrete bedding plane shear and strata failure surfaces for the range of conditions present in Australian underground coal mines. The SmartShear instrument has been built as part of ACARP project C25060.

The ability to routinely monitor roadway failure mechanisms such as shear along weak interfaces has the potential to reduce the risk associated with these hazards. The nature of ground response associated with discrete shearing along planes of weakness (at depth into the roof, floor and ribsides) is typically not manifested in the same style or magnitude that failure of...
the intact rockmass presents. It is common for little or no visual or apparent warning to precede a fall associated with shear along discrete planes of weakness. Routine shear monitoring should address these failure modes and can be used in a supplementary fashion to traditional Tell Tale based monitoring to manage strata control hazards.

**SHEAR FAILURE IN COAL MINES**

Understanding the failure of rock mass and intact rock surrounding coal mine roadways is critical for roadway stability. The inherent variable nature of rock mass, as well as continual stress redistribution during mining, means that the strata surrounding roadways is considered a dynamic environment, which can lead to strata failure. Routine monitoring of all types of roadway deformation is important for maintaining a safe and productive mining environment.

Shear failure surrounding coal mine roadways can occur along bedding planes or intact rock. Unlike tensile/dilation based failure, shear failure can have little to no visual signs at the roadway roof/rib. Shear failure generally initiates in the corners of the roadway and can propagate further up into the roof, which can lead to large scale roof failure.

**Shear failure of bedding planes in weak strata**

Commonly in Australian coal mines, weak strata surround the coal seam. Extensive research completed during ACARP project C50232 at Springvale Coal Mine found that during excavation of a roadway as the stresses are redirected about the opening, if the shear strength of the bedding plane or geological contact is overcome, failure and slip along the interface will occur. The slip along these interfaces can continue outside of the rib line and the effective span of the roadway is increased. This then leads to further redistribution of the stresses surrounding the roadway, which can increase shear failure further into the roof, and consequentially further out beyond the rib line, until an equilibrium is found. This process results in high angle shearing above the roof and beyond the ribline. During Longwall retreat, gate roads will undergo additional stress redistribution and this failure process is further exaggerated. Understanding the extent of shear failure is important, as the stability of the roadway and adjacent pillars can be compromised once the failure progresses upwards and outwards from the ribs, leading to increased effective roadway span and decreased effective pillar width.

**Shear failure of intact rock**

Shear failure of intact rock will occur once the failure criterion is met, as the stress acting on the rock overcomes the shear strength of the rock, leading to failure of the intact rock and lateral movement along the fracture (Gale 2018). As underground coal mining continues to progress deeper in Australian coal mines, the stresses surrounding roadways increase and therefore additional support and monitoring is required to overcome these additional stresses.

**CURRENT STRATA MONITORING INSTRUMENTATION**

**Tell Tales**

Currently the coal mining industry has an easy, reliable and affordable way to monitor dilation based roof deformation using Tell Tale extensometers. Multiple mechanically anchored roof extensometers are used extensively in coal mines throughout Australia. Tell Tales provide a continual visual indication of the roof conditions. They can resolve roof strata movement at up to four horizons. Tell Tales are typically installed in holes up to 10 m long for a range of hole sizes. They can be easily and quickly installed by operators on a systematic basis. (MacGregor 1998)
Tell Tales have been used in the industry for over twenty years and are considered an integral part of roadway monitoring and safety of workers underground. They are now an important part of coal mine Trigger Action Response Plans (TARPs) throughout the industry. Tell Tales are certainly an important instrument for routinely measuring dilation based deformation of a roadway, however they do not have the ability to measure shear displacement of the strata.

**Traditional Shear Strips**

Detailed underground based evaluation of shear along discrete interfaces has historically been achieved using strain gauge based shear strips as part of ACARP project (C50232). Shear strips have been used to accurately determine the location, timing, magnitude and sense of shear displacement at various Australian collieries. These are based on foil wire resistance strain gauges at close (50mm) intervals. Whilst successful at measuring high resolution of shear, they are limited to measuring shear in one dimension, have high manufacture costs and require grouting into position. This instrumentation is more suited to detailed field investigation and not for routine monitoring applications in underground coal mines.

**SMARTSHEAR**

The SmartShear is a cost effective, routine shear monitoring instrument for installation in ungrouted boreholes. The system is based on an existing MEMS sensor from Holville Pty Ltd that is certified (IECEx 12.0034X) Intrinsically Safe (IS) for use in Australian coal mines. A hand-held readout unit designed to be IS has been developed as well as an I.S. approved two wire Holville roofAlert™ communication and logging system.

The SmartShear instrument has been developed to accurately measure shear movement at multiple locations within a borehole. The shear movement is measured on a two dimensional plane at 90 degrees to the installation of the instrument using a series of MEMS tiltmeter sensors. Continual change in shear direction and magnitude over time is measured once the shear locations are detected. Figure 1 shows a schematic of the SmartShear system and the MEMS tiltmeter installed into a borehole undergoing shear movement.

![Figure 1: The SmartShear system including the MEMS tiltmeter, installed into a borehole undergoing shear movement](image)

**SMARTSHEAR DESIGN**
The SmartShear instrument consists of a series of plastic lengths known as bay lengths that are coupled together, using plastic universal joints, with one side of the joint containing a MEMS tiltmeter. Attached to each bay length is a spring loaded, plastic anchoring system that when activated, enables the bay length to centralise in the borehole, anchoring itself to hold its position. A data cable runs down the length of the instrument, attached to a motherboard which is anchored to the base of the borehole. Figure 2 shows a final breakdown of the final design of the instrument and Figure 3 shows the anchor activation system.

The MEMS tiltmeter was developed by Holville Pty Ltd and has been designed to be integrated into the SmartShear at each bay length to detect shear movement at multiple horizons throughout the borehole. The MEMS tiltmeters are developed to be pre calibrated, so that the orientation of the borehole can be determined prior to any shear movement without the need of a base borehole reading.

An intrinsically safe hand held unit is used to download the data at the push of a button. The unit is designed to be reliable, easy to use and portable. The hand held unit is shown in Figure 4. The unit is able to download data from either wired or wireless versions of the shear monitoring device.

Figure 2: The SmartShear instrument, showing a breakdown of the main components.

Figure 3: The activation and anchoring process.
FIELD TRIAL – OAKY NORTH MINE

In March 2017 a successful field trial was undertaken at Oaky North Mine. The trial involved the installation of a 10 m long SmartShear with ten MEMS tiltmeters spaced 1 m apart. The instrumentation was installed during the drivage of a longwall installation face as shown in Figure 5. The installation face was mined in two passes, with the instrumentation being installed after the first pass had been mined. After installation, shear monitoring readings were taken during widening, as the second pass was mined towards and then beyond the installation location. This provided a dynamic geotechnical environment suitable for field scale evaluation of the SmartShear and MEMS tiltmeter. The goal was to determine the location and magnitude of shear movement over time, as the second pass was mined. The effectiveness of the primary and secondary support could then be reviewed.

The SmartShear was installed so that any shear movement along the roadway was picked up as ‘pitch’ and any shear movement perpendicular to the roadway is picked up as ‘roll’. The outcome of the trial showed:

- the 10 m long SmartShear was easily installed into a 60 mm borehole
- prior to engaging the anchors, the SmartShear was able to be rotated within the borehole and then secured at the base of the borehole to the desired orientation, i.e. MEMS tiltmeters facing both parallel and perpendicular to the roadway
- successful deployment of the anchors using the deployment wire
- all anchors successfully mechanically anchored without the requirement of additional adhesion i.e. cement or resin
- successfully took readings from the SmartShear using MEMS logger and laptop. (hand held unit not fully developed by this point)
Figure 5: Plan view map of trial installation at Oaky North Mine, showing the installation location, the pitch and roll directions and the two mining passes

Oaky North Mine results

Figure 6 shows the cumulative displacement of the borehole relative to the end of the hole (as this is assumed to be the most stationary point) for both pitch and roll. The results show that the base of the borehole has moved inwards towards the second pass of mining, as shown by the positive ‘roll’ displacement. As the continuous miner mines towards the instrument location, there is a small amount of shear displacement in the lower two metres of the borehole. Once the miner mines past the installation site, the shear movement at the base of the borehole progresses further towards the second pass opening and shear horizons further up into the borehole begin to form. This trend of increasing shear movement and increasing height of shearing into the roof continues as the continuous miner mines further away from the site.

Figure 7 shows the horizontal displacement of each MEMS tiltmeters throughout the borehole relative to the above lying MEMS tiltmeter. This enables a good visual of the major shear horizons throughout the borehole.

The data can be plotted as a two dimensional plan view plot to represent the overall deviation change of the borehole from the base reading, as shown in Figure 8 referenced to the end of the borehole.
APPLICATIONS FOR SMARTSHEAR IN UNDERGROUND COAL MINES

The SmartShear has a variety of applications to resolve rock mass shear for the range of conditions present in Australian underground coal mines.

Routine monitoring:

- Life of mine development areas
- Gate roads: Monitored over the life of the gate roads to assess the shear movement during different loading periods, i.e. during drivage and additional abutment loading from longwall retreat

Campaign monitoring:
- Installation faces
- Critical infrastructure
- Support optimisation
- Determine height of shearing
- Determine high angle shear zones beyond rib line

The SmartShear instrumentation has the potential to become an integral part of the underground TARPs system. With the ability to monitor the change in shear movement over time, a range of triggers can be set depending on the location of the SmartShear within the mine as well as the geotechnical environment of the area. This will provide early indication to mining officials and operators of changing geotechnical conditions, thereby allowing early remedial action to be taken to ensure roadway stability. Valuable information gained from the SmartShear system can then be used for future roadway and support designs.

The SmartShear is designed to connect to the roofAlert system. A real time monitoring system that links up the SmartShear as well as other geotechnical instruments. The data is sent live to the control room with total movement, georeferenced instruments and acceleration alarms, prompting actions that arise from pre-set triggers within the TARPS.

OTHER APPLICATIONS

Outside the coal mining industry the SmartShear has a number of other potential geotechnical applications as follows:

- Slope stability: SmartShear can aid in monitoring the stability of slopes in open pit mining environments, civil road cuttings and potential land slide locations. With the instruments ability to pick up early signs of movement, it can be monitored in real time and connected to an alert system.
- Underground hard rock mining environments: Stress environments that are continually changing as ore is extracted from the pit. SmartShear is useful for monitoring shear movement in roadways that will be affected by ongoing stress redistribution.
- Tunnelling: SmartShear can be utilised as part of routine monitoring in civil tunnels to ensure the ground support is adequately limiting shear movement surrounding the excavation.

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REFERENCES

SHEAR STRENGTH OF ROCK JOINTS UNDER CONSTANT NORMAL LOADING CONDITIONS

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ABSTRACT: The variation of shear strength of rock joints under constant normal loading conditions was studied. Three dimensional printing technology was incorporated to produce moulds of rock joints. Rock joints samples with three different roughness values were cast using concrete with uniaxial compressive strength of 20 MPa. Samples were sheared using a direct shear testing machine for normal stress values ranging from 0.25 to 0.7 MPa. In addition, effects of shear rate on shear strength properties of rock joints were experimentally investigated. It was found that the shear strength of rock joints is a function of normal stress, joint roughness and shear rate values. In addition, it was shown that three dimensional printing technology is a useful tool to replicate real rock joints.

INTRODUCTION

Joints in a rock mass have a significant effect on the shear strength and deformation properties of the rock. Lama (1978) investigated the mechanical behaviour of a rock mass and indicated that for closely spaced joints, the mechanical performance of the rock mass is similar to the mechanical behaviour of the joints. In the past, several researchers carried out tests to explore shear behaviour of rock joints. Patton (1966), Ladanyi and Archambault (1969), Barton (1973, 1976 and 1986), Hoek (1977, 1983 and 1990), Hoek and Brown (1985), Bandos et al., (1981), Hencher (1989), Kulatilake (1993) and Saeb and Amadei (1992) performed research investigations on shear strength properties of both artificial and natural unfilled rock joints under Constant Normal Load (CNL) condition where dilation is not restricted during shearing.

Real rock joints have three dimensional roughness distributions which cannot be accurately simulated by artificial triangular or sinusoidal rock joints. In this context, Mirzaghorbanali et al., (2014) suggested that research studies on shear behaviour of rock joints should be carried out on real rock joints. Nevertheless, real rock joints with the same surface roughness value are rarely to be found in nature, thus, experiments repeatability is a challenge for researchers.

This paper describes experimental investigations into shear strength properties of joints cast using moulds of real rock joints for various normal stress and shear rate values under CNL conditions. Three dimensional printing technologies were incorporated to prepare moulds of real rock joints, facilitating shear test repeatability.

SAMPLE PREPARATION

Three moulds with different roughness values were prepared for this experiment. They were named as: 1R, 2R and 3R. As shown in Figure 1, moulds were made in pair using a blue material, incorporating three dimensional printing technologies as per direct shear testing machine specifications. Subsequently, a concrete mixture was produced and added inside the PVC moulds as shown in Figure 2(a). Once samples dried, they were taken out from moulds and left undisturbed for 28 days. Cylindrical samples were prepared using the same mixture for Uniaxial Compressive Strength (UCS) determination. USC was found to be 20 MPa after 28
days. Figure 2(b) shows one of the prepared samples. All samples had the same cross sectional diameter of 63.4 mm. This diameter was used to calculate the cross sectional area of the samples and for the shear and normal stress calculations. At the final stages, the pair of samples were positioned on each other to be placed in the shear testing machine which exerted different normal loads of 750 N, 1250 N, 1750 N and 2200 N.

![Figure 1: Moulds of rock joints prepared using three dimensional printing technology](image)

![Figure 2: (a) Sample preparation procedure (b) prepared sample based on R1 mould](image)

**TESTING PROCEDURE**

The testing machine which was used to apply the shear and normal load on the samples is ShearTrac ii as shown in Figure 3. This is an automatic loading system which includes transducers. The amount of load which this machine applies on the testing samples is controlled based on the feedback from these transducers. This machine is equipped with the sensors of two force transducers (normal and shear) and two transducers for horizontal and vertical displacement.
The system is connected to a computer to monitor the amount of load which is exerted to the samples. The computer loads or unloads the loading frame until the amount of loading which is read by the transducers becomes equal to the values required for testing the samples. Two-step motors connected to the gearing systems provide the normal and shear loads. They enable the loading mechanism to be raised and lowered for exerting the normal load and to be moved left and right for applying the shear load.

Each pair of samples was held together and mounted on the shear box in the testing machine. After calibrating the vertical and horizontal position of the sample with the normal and shear loading arms, the vertical load was set to a fixed value since the test was performed under constant normal load of 750 N, 1250 N, 1750 N and 2200 N. In the next step, the shear or horizontal load was applied and increased until the upper part of the sample slid over its lower part. It should be mentioned that the shear load was applied at a rate of 2 mm per minute for part (A) of the testing campaign.

The values of both normal and shear load and dilation results were displayed on the computer monitor during the test. This procedure was repeated for all samples under the specified loads. Shear and normal displacements were measured by the transducers and the experimental data was saved on the computer. Figure 4 indicates samples 2R when it is under a constant normal loading of 1250 N within the testing machine.
RESULTS AND DISCUSSIONS – PART (A)

Each sample was tested with different loads and results are presented in three plots: normal stress, shear stress and dilution curve. These plots are based on the variation of shear displacement. As observed in the plots, normal loads which are constant for all samples are 2200 N, 1750 N, 1250 N and 750N. In terms of the magnitude of the normal stress on the samples, load 2200 N applies a normal stress of 0.7 MPa, load 1750 N yields a normal stress of 0.55 MPa, load 1250 N a normal stress of 0.4 MPa and then the normal stress of 0.25 MPa. Shear load and stress values for samples are different due to differences in the roughness of samples. Figures 5 to 7 are the results of this test for each sample.

As shown in Figures 5 to 7, the shear stress curves first reach to their peak values with an almost linear trend and then remain almost constant as the residual stress by increasing the shear displacement. It is clear, the peak of shear stress increases with increasing the normal stress. It means, for example in the sample 1R, the peak of shear stress for 0.25 MPa normal stress is around 0.15 MPa while it is around 0.45 MPa for the normal stress of 0.7 MPa. This trend can be also observed for other samples.

Roughness of samples affects the location on which each curve reaches the peak of shear stress. For instance, for sample 1R, the peak of shear stress for different loads happens at the shear displacement of around 1.8 mm while this value decreases for samples 2R and 3R. For the same value of normal stress, it is clear that the value of roughness in different samples also affects the peak value of shear stress.

In a general trend, dilation increases with the reduction of normal stress. In other words, highest vertical load of normal stress on each sample leads to the lowest value of dilation and vice versa. However, there are some exceptions in the plots which can be due to experimental error. Negative values for dilation curves on particular shear displacement locations for some loads means the compression during the shearing load on that location and the slope direction of the rock joint is negative at those particular shear displacements.

Figure 5: Experimental results for mould 1R a) shear stress versus shear displacement b) normal stress versus shear displacement c) dilation versus shear displacement
Figure 6: Experimental results for mould 2R a) shear stress versus shear displacement b) normal stress versus shear displacement c) dilation versus shear displacement

Figure 7: Experimental results for mould 3R a) shear stress versus shear displacement b) normal stress versus shear displacement c) dilation versus shear displacement

RESULTS AND DISCUSSIONS – PART (B)

The above results were obtained with the shear rate of 2 mm per minute. In a different test series, two moulds of 1R and 3R were selected for the experiment. The same testing procedure was applied on samples cast using these two moulds with the normal load of 1750 N but shear
rate of 0.5 mm per minute and 1 mm per minute. The purpose was to investigate the effects of shear rate value on the shear stress, normal stress and dilation curve. The plots presented in Figures 8 and 9 are the results of these tests. They compare the results for samples 1R and 3R for different shear rates of 0.5 mm per minute, 1 mm/min with the previous results for these samples with a shear rate of 2 mm /min.

As indicated in Figures 8(a) and 9(a) for the shear stress versus shear displacement plot, the shear rate affects the behaviour of shear stress curve against shear displacement. Usually the peak value of shear stress increases with increasing shear rate. As the shear rate increases, the frictional resistance which is presented by the joint surface becomes larger which causes the increasing shear strength of the joint.

For the shear rate of 0.5 mm per minute, the curve first increase rapidly and then remains nearly stable at the shear stress of around 0.35 MPa. This trend for the shear rate of 1 mm per minute is similar with the difference that the curve remains nearly constant at a higher shear displacement which is due to a higher shear rate. The peak value of shear stress at this rare is 0.48 MPa. For the 2 mm per minute shear rate, the curve experiences a higher shear displacement but its peak shear stress of around 0.43 MPa is slightly less than that for the 1 mm per minute.

Figures 8 and 9 (b) present the plot of normal stress against shear displacement for different shear rates. It is clear, all curves are flat with no change in normal stress which indicated that the tests were performed under a constant normal load. There is no change in the value of normal stress at different shear rates because all experiments were carried out under the same normal load of 1750 N which applies a constant normal stress of 0.55 MPa on the samples.
Figure 9: Experimental results for mould 3R with various shear rate values a) shear stress versus shear displacement b) normal stress versus shear displacement c) dilation versus shear displacement

Dilation curves are presented in Figures 8(c) and 9(c) which is the plot of normal displacement against shear or horizontal displacement. It is obvious that all curves have an increasing trend along with the increasing shear displacement. For the shear rate of 0.5 mm per minute, the curve reaches a normal displacement (dilation) of 0.22 mm in the shear displacement of 2.7 mm. The curve related to 1 mm per minute shear rate experiences a peak normal displacement of 0.37 mm in shear displacement of 4.5 mm. For the shear rate of 2 mm per minute, the curve reached a normal displacement of 0.46 mm in a shear displacement of 7.9 mm. As a general trend for this plot, it can be said that the dilation increases with increasing shear rate.

CONCLUSIONS

The aim of this research work was to investigate the effects of roughness, normal stress and shear rate on the unfilled joint shear behaviour under constant normal load condition. Three pair of moulds with different roughness was prepared to be used in shear testing machine. Four different normal loads were selected and were applied on the samples individually. This experiment was performed in two phases. At the first stage, each pair of sample was placed on the testing machine and was applied under a constant normal load and varying shear load. The shear rate was 2 mm per minute in this phase. This process was repeated 4 times. For the next phase of this experimental work, two of samples were selected for a different experiment. The testing process was almost similar to the first phase with the difference that this stage was carried out only for one normal load and two different shear rates of 0.5 mm per minute and 1 mm per minute. Following conclusions can be made from this experiment:

- The peak shear stress increases with increasing normal stress,
- Roughness value affects the location of the peak shear stress,
- The peak shear stress increases with the increase of the joint roughness,
- Dilation increases with the decrease of normal stress,
The peak value of shear stress increases with the increase of shear rate, dilation increased with increased in shear rate, and the peak dilation decreased with reduction of roughness.

In addition, it was shown that three dimensional printing technology is a useful tool in studying shear behaviour of real rock joints.

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REFERENCES

Barton, N, 1986, Deformation phenomena in jointed rock, Geotechnique, 36(2), pp 147-167
STRENGTH PROPERTIES OF GROUT FOR STRATA REINFORCEMENT

Ali Mirzaghorbanali¹, Peter Gregor², Zainab Ebrahim, Abdullah Alfahed, Naj Aziz³ and Kevin McDougall

ABSTRACT: Past studies on the mechanical properties of grout were critically investigated and classified. Grout samples were cast using a specially designed dog bone mould. Effects of curing time on tensile strength properties were investigated, using a universal tensile testing machine for curing times intervals ranging from 1 to 28 days. In addition, various percentages of fly ash were added to cube grout samples. Fly ash mixed samples were cured for different curing times and subsequently subjected to Uniaxial Compression Testing (UCS). It was found that tensile resistance of grout samples increased with respect to curing time. Moreover, it was concluded that addition of fly ash to grout mixture increased the uniaxial compression strength of grout samples.

INTRODUCTION

Rock bolt systems were first introduced for use as mining ground supports during the late 1940’s in the form of mechanical anchoring (Mark 2017). Mechanical anchoring became the preferred support system as they weren’t subjected to stiffness weakening due to design length (Mark and Barczak 2000). As a result of the increasing popularity and widespread implementation of rock and cable bolts, numerous design variations were conceived in order to meet the explicit criteria. The development of resin and grout anchorage allowed for greater variation in rock bolt selection in order to meet specific operational requirements (Rajapakse 2008).

Cementitious grout has become a primary method of anchoring cable bolts, due to ease of installation and load transfer abilities (Mirzaghorbanali et al. 2016). Additionally, studies conducted by (Ma et al. 2013; He et al. 2015; Thenevin et al. 2017) have investigated the possible uses and mechanical properties of cementitious grouted rock-bolt systems as a method for anchoring ordinary rock bolts. Sample preparation methods proposed in these studies have adopted the grouted cable bolt installation process, and as a result, rock bolts can be susceptible to similar grout failure modes to that of cable bolts due to erroneous installation practices. Correctly installed grouted supports can provide a safe, cost effective and long-term form of reinforcement for; wedge/flake stabilisation, arching, tieback, suspension and forepoling.

Detailed studies have been conducted to determine both the mechanical properties of grouts and chemical resins (Aziz et al. 2014; Mirzaghorbanali et al. 2016) in addition to their encapsulation properties (Aziz et al. 2016). Moreover, these studies have resulted in the determination of the properties of grouts and resins for use with both cable and rock bolts as well as establishing a general practice standard.

The study conducted by Aziz et al. (2014) analysed the effects of varying resin sample properties in accordance with the various standards (ASTM C-759 1991; South African Standard (SANS1534) 2004; BS 7861 2009) to determine the effects of:
Sample shape, Sample size, Height to width or diameter ratio, Resin type, Resin age and Curing time

Samples were subjected to the testing procedures in accordance with the various standards of testing to determine:

- Uniaxial Compressive strength (UCS),
- Modulus of Elasticity,
- Shear Strength and
- Creep/rheological properties.

Additionally, Hagan et al. (2015) through the Australian Coal Association Research Scheme (ACARP C22010) investigated the effects of water to grout ratio on the UCS of both cylindrical and cube samples. This study identified a declining relationship in UCS strength with the increase of water concentrations. Moreover, when compared to cylindrical samples, cube samples achieved higher UCS values.

The study conducted by Mirzaghorbanali et al. (2016) investigated the effects of curing time on the mechanical properties of Jennmar grout BU 100 and Minova Stratabinder. Cube samples were prepared and tested at 1, 7, 14 and 28 days of curing. The study concluded that:

- The compressive strength of grout increased with curing time, and
- Both products are suitable for use in strata reinforcement.

Furthermore, the study conducted by Mirzaghorbanali et al. (2018) investigated the effects of curing time on the mechanical properties of both small and large scale samples using Minova Stratabinder. The UCS properties were determined using length /diameter ratio of 2:1 cylinder moulds of 100 mm diameter and small cube samples of 70 mm. The large-scale bending tests were conducted using prismatic moulds of 350 mm x 100 mm x 50 mm. The study concluded that:

- The smaller scale samples achieved a faster strength response to cure while large-scale samples experienced a curing delay,
- Small-scale samples achieved higher peak UCS values, and
- The four point bending test resulted in an unexpected strength weakening due to curing time.

SAMPLE PREPARATION AND EXPERIMENTAL PROCEDURE

The grout products Minova Stratabinder HS and Jennmar BU100 were selected to prepare and test samples. The Uniaxial Compressive Strength (UCS) samples were cast using the Stratabinder HS grout and fly-ash with the industry standard 50 mm cube mould. The dog-bone samples were cast using centre dimensions of 90 mm x 10 mm x 9 mm moulds. Shown in Figure 1A is the mixing equipment used to produce grout samples, Figure 1B outlines casting moulds used for the UCS samples and Figure 5 A and B shows the casting mould and cured samples for the dog-bone samples. All samples were tested using universal compression testing machines as shown in Figure 3A and Figure 6B. Samples were cast using a mixing ratio of 7 litres of water/bag and application of slight vibration to remove trapped air. All samples were stored in a controlled fog room to assist in uniform curing and finally prepared at curing times of 1, 7, 14, 21 and 28 days.

EXPERIMENTAL RESULTS
Uniaxial Compressive Strength (UCS)

To conduct the UCS tests, samples were prepared at 1, 7, 14, 21 and 28 days curing time for fly-ash contents of 5%, 10%, 15%, 20%, 25% and 30%. To ensure accuracy of the collected data each tested parameter contained six samples as shown in Figure 2A. A total of one hundred and eighty samples were produced.

The UCS values for fly-ash content at various curing times are presented in Figure 3. It is observed that while there was an overall increase in UCS for all samples, the UCS values of the samples of 10% and 15% ash content were consistently higher than those of other ash content values. The difference between the UCS of each sample was more pronounced form 14 days curing onwards. At 28 days curing the 15% samples had a UCS of 90 MPa, whereas 5% fly-ash samples only achieved 70 MPa. A comprehensive comparison was conducted and concluded that samples containing 15% fly-ash content out performed all other samples after a curing time of 21 days and is shown in Figure 4.

The observed failure mechanisms were similar to that of general grout samples and presented in three stages. In the initial stage of failure, micro-cracks were initiated. In the second stage cracks propagated. Finally, the third stage presented a complete failure as shown in Figure 2B.

Figure 1: Grout preparation [A] a view of cube moulds [B]

Figure 2: Prepared Sample 50 mm cube UCS [A], failed sample after testing [B]

The UCS values of each fly ash curing time for both the small-scale and large-scale samples are shown in Figure 3 B through F. A comparative analysis was then conducted and is presented in Figure 4 outlining the strengthening process due to the curing time. The obtained UCS values show an increase in strength over the 28 day curing period for the 10%, 15% and 30% achieving 82 MPa, 90 MPa and 80 MPa respectively. It was noted that some samples such as the 5% and 10% fly-ash concentration experienced a strength settling effect. The tested
samples reached their peak UCS at 14 and 21 days curing of 76 MPa and 86 MPa and settled to 70 MPa and 82 MPa respectively.

Figure 3: Compression tester set up [A]. UCS values at 1 to 28 curing days [B - F]
Tensile testing

Small-scale dog-bone tests were carried out on prepared samples at 1, 7, 14, 21 and 28 day curing times using both Stratabinder HS and BU100 grouts. To ensure accuracy of the collected data six samples were cast and tested for each curing time with a total of 30 samples for each grout type. Figure 6 A, B and C shows the dog-bone samples, the testing process and the sample failure respectively.

Shown in Figure 7 are the tensile failure loads at various curing times. Comparatively to the UCS tests, the tensile strength of grout increased over time with initial values of 2.5 – 2.75 MPa and 3.5 – 3.9 MPa achieved at the end of the curing period. The failure mechanism presented in the tensile tests was identical to that of the UCS tests.
DISCUSSION

The experimental study found that the UCS of composite grout samples increased with respect to curing time and achieved an optimum strength at 15% of flyash concentration of 90 MPa. Samples containing 10% fly-ash performed similarly to the 30% fly-ash samples, however the pourability of the 30% samples were greatly reduced and samples behaved similar to wet putty. Experiments indicated an increased performance to that of standard grout tests of the not 10%, 15% and 30% concentrations. While also producing the best results, the 15% samples remained fluid and easy to pour. Further studies need to be conducted on the ability to pump of the composite samples to ensure viability in the field.

Results of the tensile tests closely matched the initial expectations based on which the peak tensile load should increase with an increase in the curing time. Additionally, the Stratabinder grout performed closely to the BU100 grout while consistently achieving results 0.2 MPa greater.

CONCLUSIONS

For the purpose of this study, a new composite grout was created and tests were conducted to determine the uniaxial compressive strength. The novel grout was subject to the same testing parameters as previous cube grout studies. Additionally, ordinary Jennmar BU 100 and Minova Stratabinder HS grout samples were subjected to small-scale dog-bone tensile test. The study resulted in the following.

- Increased strength properties when comparing the novel composite grout to standard grout.
- Grout containing 15% fly-ash content achieved the highest strength at the end of the curing period.
- The novel grout maintained a similar strength to curing time relationship as standard grout.
- The increasing of fly-ash greatly impacted the viscosity of the wet grout mixture.
- When standard grout samples were subjected to tensile loading the tensile resistance increased with respect to curing time

REFERENCES


Mark, C, DESIGN OF ROOF BOLT SYSTEMS, 2017, USDo Labor, Pittsburgh Research Laboratory, Pittsburgh, PA.


OVER-CORE RECOVERY OF A DRILL STRING BOGGED IN A LONGHOLE

Frank Hungerford\textsuperscript{1} and Mitch Fagan\textsuperscript{2}

ABSTRACT: Underground in-seam directional drilling regularly encounters zones of unstable strata. In some cases, borehole collapse leads to the bogging of the drill string. With the high cost of in-hole equipment at risk and the likely loss if not recovered at the time, over-coring has become an established form of recovery. Due to in-hole friction of the over-core rods with the borehole and the bogged rods exacerbated by the deviations within a directionally drilled borehole, the depths from which over-core recovery has been successful has been limited. This paper explains the prior installations, precautions, considerations and procedures employed for a successful over-core recovery of a 717 m long drill string bogged in an in-seam borehole.

INTRODUCTION

All longhole drilling in underground coal mines is now undertaken using directional drilling to provide directional control and progressively define the borehole location relative to mine workings. Underground directional drilling regularly encounters zones of instability which must be negotiated for drilling to continue while maintaining a stable borehole. Occasionally, conditions in the borehole deteriorate rapidly and the drill string becomes bogged. With the high cost of in-hole equipment at risk in the borehole, there is an urgency to recover the equipment. Various methods of pulling, pushing and flushing with low pressure additives have been successful at releasing drill strings in the past, but when all such attempts are unsuccessful, an attempt at over-core recovery is required.

This paper presents the results of the successful over-core recovery of a drill string.

DRILLING ENVIRONMENT

The drilling environment in and around a coal seam is usually strong enough to maintain a stable borehole. However, there are numerous geological features and conditions which present strata which will either collapse immediately or deteriorate after contact with water. Features such as mylonite zones, fractured coal associated with faulting, high stress areas and complications around and through dykes have been identified as presenting instability problems with in-seam drilling (Hungerford, 1995). A bogging event is usually preceded by high fluctuations in water pressure not too dissimilar to a low angled intersection with stone at high penetration rates before the water pump stalls. Prior knowledge of what to expect in the area of drilling and driller experience are key aspects in the security of the equipment exposed to risk in the borehole.

With the high number of in-seam boreholes in place for gas drainage, an added problem has been the intersection of previously drilled boreholes and loss of water circulation into those boreholes. Water and cuttings return does not continue out the length of the borehole with the chance of cuttings accumulating at the borehole intersection. In some cases, the intersected borehole can be identified by the diverted water flow and shut to allow continued drilling. But with possible multiple connections between boreholes, this is not always successful. Preceding this event, loss of water return followed by fluctuating water pressure and surging feed are
indicators of a potential bogging event. The bogging can occur when pulling back after identifying loss of water return.

**OVER-CORING DRILLING PRACTICE**

Before any over-core operation is undertaken, a risk assessment is conducted to identify the risks and define the operational requirements of the project.

Longhole drilling had become an established practice with the conventional flip-flopping of Down Hole Motor (DHM) bend orientation for directional control. But with the practice of a change in orientation every 6 m, borehole friction is the main factor limiting borehole depth initially with directional drilling and if over-core recovery is required. In the case of over-coring, in-hole friction occurs inside the HQ over-core rods against the bogged CHD rods and externally with the sides of the reamed borehole. This combination of friction is increased with numerous deviations in the borehole and influences the depths from which over-core recovery can be considered.

An assessment can be made on the number and severity of the deviations in the borehole which will adversely affect the in-hole friction on the over-core rods. With the common surveying practice of surveying directly behind the DHM (3 m behind the bit) and changing orientation every 6 m as illustrated in Figure 6, a true indication of the severity of the deviations in the borehole is not available (Hungerford, *et al.*, 2012). When severe changes in azimuth and/or pitch are identified, notations can be made to warn the drillers of potential changes in drilling conditions. The VLI drillers in NSW have a common practice of taking a check-shot at the 3 m point between 6m survey intervals when drilling longholes. This provides more accurate control and allows analysis of the response curves to assess directional control of various bent housing and bit diameter combinations. It also allows simple analysis of borehole deviations in the case of over-coring operations.

![Figure 1: Survey positions with 6 m orientation changes](image)

Another influence of the deviations in the borehole is the effect of rotating the over-core rods while they are being flexed. HQ rods are essentially rods for wire-line coring usually used with close tolerances in boreholes with only slight deviations. When put under "knuckle joint" loading rotating through sharp deviations, failure of the male thread joint is a common occurrence.

The standpipe must be identified and altered if required to suit the proposed over-coring diameter. Most directional drilling is undertaken through a 6 m long 100 mm standpipe grouted into the rib to manage all water, cuttings and gas returns from the borehole while drilling. If a 100 mm standpipe has been installed, this standpipe has to be removed by over-coring and a 150 mm standpipe installed. This can be complicated if high gas flows are being produced from the borehole. Recent practice when drilling into new areas has been to install a 150 mm standpipe (Figure 2) which will allow over-coring if required.
HQ drill rods (88.9 mm OD x 77.7 mm ID) are a standard rod used for over-coring CHD 69.9 mm OD directional drill strings. Most drill rigs designed for directional drilling with CHD rods now have the ability to feed HQ rods through the rotation unit suitable for rotary reaming.

Over-core (shoe) bits were initially produced with Tungsten Carbide (T/C) cutters (Figure 3A). However, these bits were only suited for coal drilling and usually suffered loss of cutters which reduced the diameter being reamed. This led to an increase in in-hole friction and failed recovery attempts. The design and supply of shoe bits with Poly Crystalline Diamond (PCD) cutters provided bits which can penetrate both coal and stone and do not require checking or replacement over the length of an over-core operation. These are now provided in 98 mm diameter for cleaning out a short borehole through a 100 mm standpipe and 105 mm (Figure 3B), 115 mm and 125 mm (Figure 3C) for reaming to larger diameters in longer boreholes. The inside cutting edge of all shoe bits is stepped out from the inside diameter to ensure the cutters do not contact and potentially cut through the bogged rods.

The larger diameter reamed borehole provides reduced contact between borehole sides and outer surface of the HQ over-core rods and thus reduced in-hole friction.

Additives mixed with the flushing water are recommended to reduce the two sources of in-hole friction. A “bit lube” should be considered for the metal to metal contact between the two sets
of drill rods, which a normal borehole flushing polymer is considered for flushing of cuttings. The practice of adding unsuitable grease to the inside of the over-core rods increased the “drag” on the over-core rods and made handling difficult when pulling the recovered drill string.

The standard non-magnetic 4/5 Accu-Dril DHM fitted with a 1.25° bent housing presents a problem when completing the final stage of an over-core recovery when it is suspected that the bogging has occurred at the drill bit. The 73 mm OD of the DHM will fit within the 77.7 mm ID of the HQ rods until the over-core bit passes the bent housing 750 mm back from the front of the bit box. At a point 463 mm back from the bit box, the inside of the over-core rod makes contact with the deflected DHM (Figure 4) (REI/DPI, 2012). From there, the over-core rod is forced over the DHM with both DHM and over-core rod flexing to fit. An additional 50 mm of travel is usually required to take the over-core bit up to the back shoulder of the drill bit to eventually release the bogged drill string.

This forced fit provides a secure grip on the recovered directional drill string when pulling out of the borehole. It does present a challenge in removing the DHM from the over-core bit when removed from the borehole.

![Figure 4: Wash over comparison of HQ drill rods with 93 mm diameter DHM (REI/DPI, 2012)](image)

**RIG CAPACITY**

A track mounted VLI Series 1000 was being used for the directional drilling at Tahmoor Colliery. With a thrust capacity of 140 kN and a rotational torque capacity of 2430 Nm, the rig is one of the highest capacity being used in the coal industry and thus as adequate as any available to undertake longhole over-coring operations.

**THE BOGGING INCIDENT IN THE DIRECTIONAL DRILLED BOREHOLE**

Management at Tahmoor instructed VLI to drill a 500 m long borehole (TGW1-5CT-D12) from which to core for gas drainage compliance assessment adjacent to the proposed development of Tailgate TGW1 (Figure 5). A standard Asahi 96.1 mm diameter PCD bit was used. The B Heading of the tailgate in the new western domain had previously been flanked by two 1600m long gas drainage boreholes which had been drilled from 2CT, B Heading. When the compliance cores at 271.5 m, 385.5 m and 500 m (Figure 6) indicated drainage had been insufficient in the area, the mine requested the 500 m long borehole be extended to a depth of 1000 m. The design of the extension was to turn left and cross one of the previously drilled 1200 m long boreholes (at approximately 726 m) before continuing parallel to the proposed rib-line (Figure 7).

Two zones of soft coal at 630 m and 653 m (Figure 6) were identified with the drilling. Otherwise, the coal drilling was stable.

When at a borehole depth of 819 m, the driller noticed the return water flow had reduced by 50%. Extended flushing re-established full return flow and allowed drilling to continue to 837 m before water return reduced again. After a change of shift, water return had been lost
Figure 5: Borehole TGW1-5CT-D12 location

Figure 6: Borehole profile

Figure 7: Lateral deviation
Figure 8: Profile of bogged rods

completely. The rods were pulled back to a depth of 717 m before the drill string become bogged (Figure 8). With a branch point at 720 m, it was thought that dumping of cuttings in the enlarged annulus of the branch point had likely bogged the rods at the bit. The crossing of the previous longhole at approximately 726 m was possible source of instability or water loss dumping cuttings in the borehole. Soft coal had been recorded at 630 m and 653 m during the directional drilling but was not thought to be responsible for the bogging.

The mine’s pressure water was connected to the drill string and all boreholes in the area were checked for water flow then shut off. With no identified connection and no subsequent water return, the decision was made to over-core the bogged drill string.

OVER-CORING DRILLING PRACTICE

Before drilling commenced, the drillers were instructed on the practices required for the recovery operation. The initial drilling practices included:

- Rotational speed of 120 to 150 rpm.
- 240 to 250 l/min water flow rate.
- Record usual drilling parameters of thrust and hold-back hydraulic pressures and water pressure and flow rate.
- Record main hydraulic pump pressure as an indication of the rotary drilling pressure.
- Measure and record the rotational speed.

With a 100mm standpipe installed for the initial drilling, the standpipe was removed by over-coring with HWT casing rods and a 3 m long 150 mm standpipe installed.

With the length of the bogged string of 717 m near the likely maximum over-core recovery depth, it was decided to use the maximum available shoe bit diameter of 145 mm for the initial reaming pass.

With the drillers recording check-shot surveys at 3m intervals, the changes in both azimuth and pitch can be accurately assessed. An analysis of these changes in azimuth (Figure 9) and
changes in pitch (Figure 10) did not identify any significant changes in either which would have adversely affected the over-coring operation. The ranges of changes were within normal response curves for directional drilling.

The on-site team was also directed to use an “anti-chatter” grease between the rods meant to reduce the metal to metal friction.

The borehole was reamed with progressively increasing torque resistance (Figure 9) until rotation and penetration could not be continued beyond a depth of 633 m. The hydraulic system (rotation) pressure was just below that listed as the maximum available but had plateaued out at 23.6 MPa. The borehole was cleaned out with numerous rotary flushing passes without improving the frictional characteristics of the borehole. The grease between the rods was
deemed as adding friction to the system due to the binding of the over-core rods when being removed from over the bogged CHD rods. The use of the grease was discontinued.

In an attempt to reduce in-hole friction, the shoe bit manufacturer (Hard Metal Industries) was asked to modify a standard 125 mm diameter bit to increase the reaming diameter to 145 mm. With this modified shoe bit, the borehole was reamed to a depth of 444 m before there was a noticeable reduction in the flow of flushing water returning from the borehole. It was thought that some water was passing through the bogged rods and disappearing into an adjacent borehole. A blanked CHD box to pin-sub was attached to the back of the bogged drill string which re-established full return water flow. The threaded blank was used so access was possible if the drill string was pumped down the rods on release.

Reaming continued to 600 m before further rotation was not possible significantly below the hydraulic pressures achieved with the initial 125 mm reaming. Up to that depth, the frictional load of inter-rod friction and reaming from 125 mm to 145 mm was equivalent to that of reaming from 96 mm to 125 mm (Figure 9).

Several problems had started to develop with the operation of the drill rig during both the reaming and cleaning out operations. The hydraulic systems on the drill rig were checked and reset to ensure the rig was operating at its optimum for the next stage of reaming.

The 125 mm shoe bit was flushed back into the original face at 633 m. From there, reaming continued with water flow reduced to 100 l/min and back-flushed each rod with a water flow of 200 l/min. After reaming to 687 m, the internal CHD rods were no longer visible inside the HQ over-core rods. CHD rods were fed down inside the HQ rods to contact and connect to the blank sub. The bogged rods had been pumped back to the 837 m depth of the directional drilling. The hydraulic pressures were only slightly lower or equal to those recorded at the limit of the initial 125 mm reaming pass (Figure 9) but rotation was being comfortably maintained.

No instability, structure or borehole crossing was located in the vicinity of the bogging zone. Both sets of drill rods were successfully removed from the borehole.

![Figure 11: Rotational pressure of over-core drilling](image-url)
CONCLUSIONS

Directional drilling relies on a combination of prior knowledge of the conditions expected in an area of proposed drilling and the experience and awareness of the driller to respond to a potential bogging incident. Quick response prevents numerous bogging of drill rods during the normal course of directional drilling projects. But some bogging events occur in spite of the precautions and experience. Good directional control while drilling the initial borehole provides characteristics which more easily facilitate the over-coring reaming. These include:

- Good record keeping of in-hole strata intersected and conditions.
- Regular surveying which includes surveys at each change of DHM orientation.
- Drilling with the minimum DHM deflection which provides adequate directional control to minimise deflections in the borehole.

A successful over-core recovery then relies on a risk assessment to assess the conditions and determine the most likely course of action required to recover the bogged drill string. Experience from previous recovery operations plays a big part of this assessment.

Due to the expensive equipment at risk when directional drilling in variable conditions, it is advisable to have a dedicated kit of recovery equipment so the operations can commence without long delays and with confidence the equipment is fit for purpose and in good condition.

Then the drillers involved need to be supplied with support information defining the recovery project and any drilling parameters and flushing additives to use. Good record keeping from this point allows a progressive assessment of the progress of the reaming and the likelihood of the initial procedures being successful. Timely adjustments and changes can be made to improve the chances of success.

REFERENCES


FLOW MEASUREMENT IN GAS DRAINAGE

Tim Harvey¹, Ian Gray²

ABSTRACT: This paper reviews the need for flow monitoring in gas drainage and the flowmeters that are available to do this. As suitable flowmeters that are capable of dealing with gas, water and particulate matter do not exist work has been done to adapt ball valves for this purpose. They have been found to fill the function admirably and with a combined manometer and pressure sensor can provide flow ranges in excess of 100:1. They can also be left wide open to avoid blockage when not being used for measurement.

INTRODUCTION

Flow measurement in a gas drainage operation in a coal mine is necessary for the determination of the gas content of an area using a material budget calculation as shown in Equation 1:

\[ \text{Gas in Place} = \text{Gas Initially in Place} - \text{Gas Drained} + \text{Gas Sources} \]  

Where

- Gas Initially in Place is the coal’s gas content x density x coal volume in the area being considered
- Gas drained is the gas that is drained out of the area being considered and should include that drained from boreholes as well as that lost into the ventilation system.
- Gas Sources is gas recharge (usually from roof or floor) or could be unmeasured losses from the area being considered.

Borehole flow measurement is usually the most important measurement in determining the Gas Drained term. Its measurement is generally infrequent and poorly done. The determination of gas lost in ventilation should be determined by monitoring airflow in roadways and sampling for gas content changes in the airstream at ends of the roadway.

The Gas Sources term may be significant and should not be ignored. They are the reason why material balance calculations may be significantly in error. They must be determined by checks on gas content during drainage. These may be point checks by drilling and sampling or, more effectively, by the installation of permanent pressure sensing points, which may be continuously monitored. The latter may be installed from underground (Gray 1987) or from surface in the form of piezometers (Gray and Neels, 2015). The relationship between gas pressure and gas content should however be determined by Native Isotherms (Gray, Wood, Shelukina and Zhao 2015) rather than theoretical combinations of individual gas isotherms as the latter are frequently inaccurate.

The determination of whether a hole is flowing or not and whether it is flowing at half the rate of adjacent holes may be important indicators as to whether it is blocked or not.

Measuring gas flow in an underground environment is difficult and most flow meters are unsuitable for this purpose. This paper presents a simple adaptation of ball valves which enables them to be used for flow metering.

REQUIREMENTS OF FLOW METERS

Gas flow meters work best if they have a clean, dry gas that flows evenly.

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In reality the gas coming from underground boreholes is generally water saturated, contains droplets of salty water and periodically includes ejection of some particulate matter. Gas flow may be interspersed with slugs of water flow. The particulate matter can consist of lumps of coal that may be of 20 mm size being ejected at 20 m/s in some high flow holes.

The obvious way to deal with this would be to place a water separator on each hole and to meter gas flow downstream of the separator. This, however, poses a significant cost, complexity and space constraint on the underground operation. In addition the separator would need to be protected from particulate matter becoming jammed in its entry or exit. This could be achieved by the use of stone traps. The use of stone traps before individual hole gas-water separators and flowmeters is not accepted practice. Separators that are large and therefore have large inlets and outlets can accommodate particulate matter. These are usually placed downstream of any individual hole monitoring. Having a single separator at a gas drainage stub reduces complexity but still poses a significant maintenance problem that is often not properly handled by the mine.

Without protection from water and particulate matter any flowmeter that includes a permanent restriction to full bore flow will partially or fully clog and either give inaccurate readings or block flow.

The use of electronics in coal mines poses significant complications because of the need to meet intrinsically safe requirements. This eliminates most commercial flowmeters; even if they have one of the international IS certifications, because of specific state by state requirements within Australia.

The nature of gas drainage does not however require continuous on line monitoring. Typical requirements would be monitoring daily for the first few days and while drilling continues in the gas drainage stub. This would then be spread out to weekly for the next four weeks and then monthly. This means a gas drainage hole typically only needs about 30 flow measurements during its entire operational life.

An underground flow meter needs to be:

- Reliable and accurate to +/-5%;
- Able to measure over a wide range of flow rate;
- Able to monitor flow streams containing gas, water, solids;
- Cope with Intermittent gas flow with slugs of water;
- Not have any permanent partial blockage to flow

**CURRENT PRACTISE**

The current usage of flow meters varies depending on whether the installation is underground or on the surface.

**Underground**

In underground gas drainage of mines, the current flow measurement practice varies but may include:

- installing flow measuring devices on every drainage hole;
- measuring total flow from drill stub;
- measuring flow on surface at the top of gas risers;
• no measurement at all; and
• a combination of any of the above.

The frequency of measurement varies and includes continuously, weekly or fortnightly, when the staff remembers and when staff have the time to carry it out.

The most commonly used flow meter in underground gas drainage is the differential pressure orifice meter with a removable orifice plate. These orifice plates cannot be left in the pipework so a measurement means opening a Victaulic joint, inserting the orifice plate, closing the joint and then taking a pressure and differential pressure measurement. Following this the orifice plate needs to be extracted by the reverse procedure. This is time consuming and leads to leakage from or into the gas pipelines. Venturi meters have also been used.

The flow environment, with coal particles as well as gas means that anything that restricts flow is likely to lead to a blockage. Line pressure and differential pressure are at times measured with an approved electronic instrument, a U-tube manometer or Magnehelic gauges. The latter generally succumb in a few days to the wet and corrosive gas that are encountered in gas drainage operations.

Other meter types and instrumentation may be used but the short duration, IS and communications issues means they are generally impractical for underground gas drainage installations.

**Surface**

Traditionally surface gas drainage installations have used venturi meters, orifice plates and other differential pressure devices. Any one of these that have upstream facing ports such as pitot tubes, and Annubar meters suffer from blockage. Some installations have used turbine flowmeters. More recently ultrasonic flow meters are becoming more popular as prices have reduced. These have both a reasonably wide measurement range and potentially higher accuracy. They also have the advantages of being non-intrusive, with no moving or wearing parts and retain their calibration settings.

**FLOW METER TYPES**

This section describes the various types of flow meters. Some of the information in this section is drawn directly from Crabtree 2009.

**Differential pressure meters**

Differential pressure flow meters encompass a wide variety of meter types that include: orifice plates, venturi tubes, nozzles, wedge, venturi-cone, Annubar, Pitot tube, elbow and variable area devices. Differential pressure meters are the most widely used technology for flow measurement. With the exception of the Pitot tube which measures the difference between the stagnation pressure and the static pressure of a thin probe, or simply monitoring a pressure drop over a long length of pipe, all other differential pressure devices involve placing some restriction to flow in the pipework.

The flowrate is determined from a combination of the continuity and Bernoulli’s equations. The general equation describing pressure drop across a differential pressure device such as an orifice plate is given in Equation 2 (Streeter, Wylie and Bedford, 1998).

\[ Q = C A_0 \sqrt{\frac{2\Delta p}{\rho}} \]  

where \( C \) is a coefficient depending on the geometry and Reynolds number of the flow

\( A_0 \) is the area through the flow orifice
\( \Delta p \) is the pressure drop across the device
\( Q \) is the flow rate at the density of the fluid
\( \rho \) is the density of the fluid

The nature of Equation 2 means that the pressure drop rises with the square of the flow rate. This limits the flow range to about 4:1 with a single differential pressure measurement sensor. Ranges may be extended by using high and low range differential pressure sensors. Two of these may extend the range to 8:1.

The Sigra ball valve flow meter is a differential flow device with the important difference that the flow restriction is temporary and the flow area is variable.

**Mechanical meters**

There are two types of mechanical meters these are positive displacement and turbine meters.

- Positive displacement meters have several different types including
  - sliding vane
  - oval gear meters
  - lobed impellers
  - oscillating pistons
  - nutating disc
  - fluted rotor meters
  - wet-type gas meters.

These are highly accurate and are generally used for custody transfer applications of clean fluids. Because of their mechanical nature they are subject to wear and have relatively high pressure losses. Their use is limited to low volume applications and they are not suitable for high or low viscosity fluids.

Turbine meters are used in both for both gas and liquid flow. They have a high range-ability that gives them a measurement ratio of up to 20:1. They can operate at very high pressures and are available to suit a wide range of pipe sizes. Turbine meters have have high accuracy (+/- 0.5%) and excellent repeatability. They can be suitable for very low flows. At higher flows, cavitation can be an issue. They are subject to pressure losses of up to 30 kPa at maximums flows and bearing and impeller wear needs to be monitored. Propeller, and impeller meters also fall in this class.

All these devices place a restriction in the flow path and can be easily damaged by particles.

**Oscillatory flow meters (Vortex meters)**

Oscillatory flow measurement systems involve three primary metering principles: vortex, vortex swirl (a commercial device) and the Coanda effect. In all three, the primary device generates an oscillatory motion of the fluid whose frequency is detected by a secondary measuring device to produce an output signal that is proportional to fluid velocity.

Vortex meters use the presence of a bluff body within the conduit to generate vortices that are shed behind it. The frequency of shedding is proportional to the flow rate and their calibration does not change with time. The limits are determined at the low-end by viscosity effects and at the upper end by cavitation or compressibility. Vortex shedding flowmeters are highly linear but need to operate at high flow rates with Reynolds numbers greater than 30 000. They are not suitable for low gas pressures. The flow range for gases is about 20:1.
The presence of a bluff body within the flow stream means that vortex shedding devices may cause jams of particulate matter in dirty gas streams.

**Electro-Magnetic Flow meters**

Electro-magnetic flow meters are only suitable for ionic liquids and not gases. Apart from this major limitation in gas drainage applications they are admirable devices with which to measure liquid flow.

**Ultrasonic flow meters**

Ultrasonic flowmeters are suitable for both liquids and gases. They are a non-intrusive measuring device which can be used on a wide range of pipe sizes from 75 to 3000mm with accuracies of around 1% and have range-ability up to 125:1 for higher quality meters. The three basic principles used in ultrasonic metering are the Doppler method, the time-of-flight method and the frequency difference method.

Apart from not obstructing the flow, ultrasonic flowmeters are not affected by corrosion, erosion or viscosity. Most ultrasonic flowmeters can be bi-directional.

**Advantages**

- Suitable for large diameter pipes,
- No obstructions, no pressure loss,
- No moving parts, long operating life,
- Fast response,
- Weld-on transducers may be installed on existing pipe-lines,
- Multi-beam systems can be used to eliminate the effects of flow profile, and
- Not affected by fluid properties.

**Disadvantages**

- In single-beam meters the accuracy is dependent on flow profile,
- Fluid must be acoustically transparent,
- Expensive, and
- The pipeline must be full – not a problem with gases.

**Application limitations**

For the transit time meter, the ultrasonic signal is required to traverse across the flow, therefore the liquid must be relatively free of solids and air bubbles. Anything media with a density different to the process fluid will affect the ultrasonic signal.

**Coriolis meter**

Coriolis meters are used for high accuracy mass flow measurement. Apart from custody transfer applications they are used for chemical processes and expensive fluid handling.

**Advantages**

Some of the many benefits include:
• direct, in-line and accurate mass flow measurement of both liquids and gases,
• accuracies as high as 0.1% for liquids and 0.5% for gases,
• mass flow measurement ranges cover from less than 5 g/m to more than 350 tons/hr,
• measurement independent of temperature, pressure, viscosity, conductivity and density of the medium,
• direct, in-line and accurate density measurement of both liquids and gases,
• mass flow, density and temperature can be accessed from the one sensor, and
• can be used for almost any application irrespective of the density of the process.

Drawbacks
• expensive,
• many models are affected by vibration,
• current technology limits the upper pipeline diameter to 150 mm, and
• secondary containment is sometimes necessary in case of pipe failure.

Thermal gas flow meters
These devices involve electrically heating an element within the flow stream and measuring the temperature of it or around it. These flowmeters are highly accurate for use with clean gasses and can operate with flow ranges up to 150:1. The need for a clean gas means that they are unsuitable for gas drainage flow measurement. The characteristics of various flowmeters is summarised in Table 1.

REQUIREMENTS FOR DIFFERENTIAL PRESSURE MEASUREMENT
To accurately calculate the gas flow in a differential pressure flow meter requires:
• Absolute pressure measurement
• Differential pressure measurement across the flow obstruction
• An obstruction of known form and properties
• Pipe diameter
• Gas temperature
• Gas properties – density, viscosity, cp/cv

The accuracy of absolute pressure measurement needs to the nearest kPa and the differential pressure measurement needs to be as accurate as possible, as does the measurements of diameters and areas. Figure 1 gives a comparison of the accuracy of some of instruments used for differential pressure measurements.
**Table 1: Summary of flow meter types**

<table>
<thead>
<tr>
<th>Differential Pressure</th>
<th>Dirt blockage</th>
<th>Flow range</th>
<th>Accuracy %</th>
<th>Cost</th>
<th>IS required</th>
<th>Fluid type</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orifice Plate (per plate)</td>
<td>X *</td>
<td>4:1</td>
<td>2</td>
<td>Low if not instrumented</td>
<td>If instrumented</td>
<td>Clean fluids only</td>
<td>Moderate loss</td>
</tr>
<tr>
<td>Orifice plate (multiple)</td>
<td>X *</td>
<td>&lt;100:1</td>
<td>2</td>
<td>Low if not instrumented</td>
<td>If instrumented</td>
<td>Non viscous fluids</td>
<td>Moderate loss</td>
</tr>
<tr>
<td>Sigra ball valve flow meter</td>
<td>□</td>
<td>&gt;100:1</td>
<td>2</td>
<td>Low</td>
<td>No</td>
<td>Non viscous fluids</td>
<td>Moderate loss</td>
</tr>
<tr>
<td>Venturi</td>
<td>□</td>
<td>4:1</td>
<td>0.75</td>
<td>Low if not instrumented</td>
<td>If instrumented</td>
<td>Non viscous fluids</td>
<td>Low loss</td>
</tr>
<tr>
<td>Wedge meter</td>
<td>OK</td>
<td>5:1</td>
<td>2</td>
<td>Low if not instrumented</td>
<td>If instrumented</td>
<td>Non viscous fluids</td>
<td>Medium loss</td>
</tr>
<tr>
<td>V-Cone</td>
<td>OK</td>
<td>10:1</td>
<td>0.5</td>
<td>Medium</td>
<td>Yes</td>
<td>Non viscous fluids</td>
<td>Medium Loss</td>
</tr>
<tr>
<td>Pilot tube or Annubar</td>
<td>X</td>
<td>3:1</td>
<td>1</td>
<td>Low if not instrumented</td>
<td>If instrumented</td>
<td>Clean Fluids</td>
<td>Low Loss</td>
</tr>
<tr>
<td>Variable area</td>
<td>X</td>
<td>10:1</td>
<td>1</td>
<td>Low</td>
<td>No</td>
<td>All fluids</td>
<td>Low flows only</td>
</tr>
<tr>
<td>Other devices</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultrasonic</td>
<td>X</td>
<td>up to 125:1</td>
<td>0.2</td>
<td>High for large</td>
<td>Yes</td>
<td>Clean fluids</td>
<td>No Loss</td>
</tr>
<tr>
<td>Vortex shedding</td>
<td>X</td>
<td>30:1 gases</td>
<td>1</td>
<td>Medium</td>
<td>Yes</td>
<td>All fluids (cleanish)</td>
<td>Low loss</td>
</tr>
<tr>
<td>Turbine</td>
<td>X</td>
<td>20:1</td>
<td>0.2</td>
<td>Medium</td>
<td>If instrumented</td>
<td>Clean fluids only</td>
<td>Low loss</td>
</tr>
<tr>
<td>Positive displacement</td>
<td>X</td>
<td>up to 100:1</td>
<td>0.5</td>
<td>Wide range</td>
<td>If instrumented</td>
<td>Clean liquids or gas</td>
<td>High loss</td>
</tr>
<tr>
<td>Magnetic</td>
<td>some</td>
<td>&gt;30:1</td>
<td>0.2</td>
<td>Low</td>
<td>Yes</td>
<td>Liquids only</td>
<td>No loss</td>
</tr>
<tr>
<td>Coriolis meters</td>
<td>□</td>
<td>&lt;=100:1</td>
<td>0.5</td>
<td>High</td>
<td>Yes</td>
<td>All fluids</td>
<td>To 150 mm diameter</td>
</tr>
<tr>
<td>Thermal mass flow</td>
<td>X</td>
<td>up to 150:1</td>
<td>0.2</td>
<td>Medium</td>
<td>Yes</td>
<td>Gas</td>
<td>Low loss</td>
</tr>
</tbody>
</table>

*Blockage is likely to occur if the orifice is left in place, ** IS refers to the need for intrinsically safe electronics
Note: The 3051CD is a Rosemount differential Pressure Transducer, 3051SMC Ultra is a Rosemount MultiVariable transmitter, the Comark C9501S is an IS portable pressure Instrument and the data for the U tube manometer assumes that this instrument can be read to 1 mm of water head.

SIGRA FLOWMETER DEVELOPMENT

The problem of gas flow measurement underground is not trivial and requires a quite specialised solution. In the author’s view the commonly adopted practise of opening Victaulic couplings to enable the temporary insertion of an orifice plate for flow measurement is completely unsatisfactory both from time and safety viewpoints.

What is needed is a low cost precision flowmeter with a wide flow range that poses no obstruction to flow when not in use. The low cost aspect of this means minimal or no machining or fabrication while the high flow range means doing something different from a standard differential pressure device with a maximum to minimum flow ratio of 4:1.

Several ball valves have been tested as flow meters and been found to be suitable. They can be left fully open when not being used for flow metering. When being used to measure flow they can be partially closed to form an adjustable area orifice. In addition they may be used as a shut in valve on the standpipe or in the conduit from the standpipe if the borehole needs to be closed in.

The ball valves have been calibrated using air against a range of standard orifice plates using water filled manometers to determine the difference in upstream line pressure from atmospheric and the pressure difference across the orifice plate. The air pressure and humidity was determined from an adjacent weather station and the line air temperature was directly measured. The testing was accomplished while drawing air through the flow meters and then in discharging from the blowers into the meters. In the latter case the temperature and inlet pressure were significantly different.

The calibration constant derived for each angle of opening of each valve was the term $C_{Ao}$ in Equation 2. The value of $C$ is a complex function of the inlet pipe size to valve area ratio open (dependent on opening angle) and the Reynolds number of the inlet flow. The latter is not however a critical factor in the flow calculation.

The calibration for one type of ball valve flowmeter is shown in Figure 2. The value of $C_{Ao}$ is made dimensionless by dividing by the pipe cross sectional area. The exact values for this curve are dependent on the ball valve design.

![Figure 2: Value of $C_{Ao}$/Pipe Area for a ball valve](image-url)
A photograph of a ball valve fitted with a protractor by which to measure the valve opening is shown in Figure 3.

A water filled manometer is difficult to transport, fill and read underground. Therefore an alternative simple device has been constructed to measure the gauge flowing line pressure and the differential pressure. It is not electronic and therefore avoids intrinsic safety issues. We have named it a Bubbleometer. A drawing of it and the Ball Valve Flowmeter is shown in Figure 4. It comprises water filled cylinder in which are installed two tubes and a slide valve.

Using the Bubbleometer requires connecting it to the Ball Valve Flowmeter with the tapping valves closed. The slide valve is then moved to connect the 50 mm tube to the upstream tapping while the cylinder is connected to the downstream pressure. The valve is then gradually closed until bubbles just begin to be emitted from the base of the clear 50 mm tube. This is easily seen as the meniscus is gradually pushed down the tube. If the flow is high and the angle of closure is too low to be reliable the slide valve can be moved to the 500 mm tube and the ball valve closed further so as to just produce a bubble. The use of bubbles enables the differential pressure to be measured to 0.5 mm water gauge accuracy.

The gauge pressure may then be determined by moving the slide valve to connect the 500 mm tube to atmospheric pressure and then moving it to connect the upstream tapping. The compression or expansion of the gas within the tube may be measured on the scale within the cylinder and can then be used to calculate the gauge pressure.

![Figure 3: Ball Valve Flowmeter with tapping and fitted with protractor to measure opening angle](image3)

![Figure 4: Ball valve flowmeter and bubbleometer](image4)
The determination of the flow rate requires knowledge of

- The local atmospheric pressure
- The gas composition - taken from a bag sample
- The gas temperature – measured very occasionally at the borehole collar and normally equal to seam and seam water temperature
- The gauge pressure - determined from the reading of gauge pressure in the Bubbleometer
- The Bubbleometer tube used (50 or 500 mm immersion)
- The valve angle

Using an assumption of 100 % humidity the STP gas flow may be readily calculated.

**USE OF THE BALL VALVE FLOWMETER AND BUBBLEOMETER**

The method of use envisaged for the flowmeter is that the valve on each standpipe should be replaced by a Ball Valve Flowmeter which then becomes the shut off valve for the borehole. If the drainage hole forms part of a gas drainage stub then a Bubbleometer should be hung in each gas drainage stub and connected to each flowmeter in turn to obtain the requisite measurements for flow determination.

Flow measurement in this underground situation will always be compromised in the initial stages of gas production by slugs of water. There is nothing that can be done except to wait until these have passed when taking a measurement. Other lesser fluctuations in flow may be able to be damped by fitting dead volumes (tubes) and flow restricting nozzles to the connections between the Ball Valve Flowmeter and the Bubbleometer. This has not yet been tested.

It should be noted that ball valves should not be permanently left in the partially closed position because of wear on the ball and the seals.

**CONCLUSIONS**

The need for gas flow measurement in gas drainage is presented in this paper. It forms an important part of material budget calculation of gas in place. It also enables such elementary problems as borehole blockage to be detected.

Because of the unique conditions of gas flow measurement underground a simple high range (100:1) flowmeter has been developed from a ball valve for this purpose. This has the advantage that it can be used as a shut off valve on the borehole. Because only about 30 measurements are required in the life of a gas drainage hole to be able to measure the gas it produces a flowmeter that does not operate most of the time is required. The ball valve is a precision machined device which can remain open most of the time thus avoiding problems with wear and blockage. The Ball Valve Flowmeter may be used with a variety of electronic pressure and differential pressure measurement devices. However the method suggested here is to use a non-electronic device which enables the precise measurement of gauge and differential pressure. This has been named the Bubblometer.

The Ball Valve Flowmeter can be readily obtained in 25, 50, 80 and 100 nominal bore sizes. It may be used in other applications which require the periodic measurement of dirty gas flows.
REFERENCES


APPLICATION OF VENTSIM TO LOW PRESSURE GAS DRAINAGE AND HIGH PRESSURE NITROGEN RETICULATION SYSTEMS

Roy Moreby¹

ABSTRACT This paper outlines a methodology employed to apply Ventsim ventilation modelling software to gas drainage and nitrogen injection systems. Use of the software to account for compressible flow, specified gas compositions and the resistance of conduits of various dimensions makes it suitable for both ventilation circuits and gas reticulation systems. Unlike mine ventilation circuits, in gas drainage systems it is necessary to specify flow rates arising from gas pre drainage holes as these are dependent on gas reservoir properties. This requires a mass flow balance to be obtained between underground gas drainage holes and that reporting to surface pump(s). In nitrogen injection systems it is convenient to create an interface between the gas drainage reticulation system and the mine workings which in turn is connected to the ventilation system. However, it is also possible to keep the two systems separate for simplicity and tuning. The results of applying these strategies to an operating gas pre drainage system are described. These provide the “as built” pipe friction factors obtained by matching the model results to observed differential pressures. The Pike River re-entry nitrogen injection system has also been modelled. This uses the same principles as the negative pressure gas drainage models but with high positive pressures of up to 1,000 kPa (10bar).

BACKGROUND THEORY

The mathematical theory describing frictional losses in gas reticulation systems is provided by, amongst others, McPherson, (1993) and Boxho, et al., (2009). In summary, the calculation proceeds as follows;

1. Calculate the “rationale “resistance of the pipe from dimensions and wall roughness factor. This being the physical resistance of the pipe without a correction for density.
2. For a defined gas composition, calculate the gas constant (J/(kg.K)) and density from which the mass flow rate (kg/s) can be calculated for a given gas flow rate (m³/s at specified temperature and pressure).
3. Calculate the change in pressure due to frictional losses using gas laws corrected for compression or expansion of the gas mixture.

The form of the final equation for pressure loss in a horizontal pipe with compression is shown in Equation 1 with that for the rational resistance in Equation 2.

\[
P_{1}^{2} - P_{2}^{2} = \frac{M^{2}}{2} \cdot R \cdot T \cdot \frac{r}{n}
\]

Eqn.1

\[
r = \frac{64 \cdot f \cdot L}{2 \cdot n^{2} \cdot d^{5}}
\]

Eqn.2

where,

P₁ and P₂ = absolute pressures at the start and end of the pipe, (Pa)
M = mass flow rate of the gas mixture, (kg/s)
R = gas constant for the gas mixture, (J/(kg.K))
T = absolute temperature, (K)
n = rationale resistance of the pipe, (m⁻⁴)
f = dimensionless friction factor
L = pipe length (m)
d = pipe diameter (m)

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For design of gas reticulation systems, as with Atkinson’s equation for frictional losses in ventilation systems, the following issues need to be considered;

1. Resistance is inversely proportional to the pipe diameter to the fifth power i.e. small changes in diameter have a profound effect on frictional losses. Appropriate sizing of pipes is critical.
2. For pipes under negative pressure, increasing suction pressure decreases absolute pressure in the pipe resulting in a reduction in density and a consequential increase in volumetric flow rate i.e. the volumetric flow rate increases with increasing suction. The opposite is the case in a pressurised system which is why injection pipes are significantly smaller than drainage pipes.
3. The frictional loss is proportional to the square of the mass flow rate i.e. double mass flow rate results in four times the frictional loss. Other than limiting flow rates in a single pipe this also means that installing parallel pipes, or ring mains, provides a reduction in resistance inversely proportional to the square of the number of parallel paths.

These calculations can be undertaken using a spreadsheet approach but are now also provided for in Ventsim models. It is recommended that both methods are employed to double check results prior to selecting critical infrastructure of this type.

**VENTSIM SETTINGS**

Due to the low flow rates and high pressure differentials occurring in gas drainage or nitrogen injection systems, compared to those in ventilation circuits, it is necessary to modify Ventsim simulation settings to avoid simulation errors. The custom settings to change are shown in Table 1:. These are mainly focused on increasing simulation precision, even if the simulation time increases.

**Table 1: Ventsim settings for gas reticulation**

<table>
<thead>
<tr>
<th>Ventsim Setting</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Presets</strong></td>
<td></td>
</tr>
<tr>
<td>Resistance (at 1.2 kg/m$^3$)</td>
<td>for items such as bends or valves, leakage path air to hole collar and pipe joints</td>
</tr>
<tr>
<td>Friction factor (at 1.2 kg/m$^3$)</td>
<td>for each pipe diameter only use custom for individual pipes as a last resort one for each pipe diameter for faster editing as a group set for hole collars or nitrogen plant outlet (note ambient intake will be fresh air)</td>
</tr>
<tr>
<td>Layers Air Type</td>
<td></td>
</tr>
<tr>
<td>Gas mixtures</td>
<td></td>
</tr>
<tr>
<td><strong>Simulation - Airflow</strong></td>
<td></td>
</tr>
<tr>
<td>Allowable error</td>
<td>0.001 m$^3$/s for 1.0 l/s</td>
</tr>
<tr>
<td>Compressible flow</td>
<td>Yes</td>
</tr>
<tr>
<td>Convergence limit</td>
<td>0.100 m$^3$/s or lower if pressure excess problem on convergence</td>
</tr>
<tr>
<td>Density adjust friction factors</td>
<td>Yes</td>
</tr>
<tr>
<td>Density adjust pre set resistance</td>
<td>Yes</td>
</tr>
<tr>
<td>Maximum simulation pressure</td>
<td>set 10 x maximum e.g. for gas drainage +1,000 kPa for N2 injection 10,000 kPa</td>
</tr>
<tr>
<td>Use Natural Ventilation Pressure</td>
<td>optional if 3D model</td>
</tr>
<tr>
<td><strong>Simulation - Gas</strong></td>
<td></td>
</tr>
<tr>
<td>Use gas density for air simulation</td>
<td>Yes</td>
</tr>
<tr>
<td><strong>Simulation - Environment</strong></td>
<td></td>
</tr>
<tr>
<td>Pressure</td>
<td>for ambient or seam intake or NP (101.3 kPa)</td>
</tr>
<tr>
<td>Temperature</td>
<td>for ambient or seam intake or NT (20°C)</td>
</tr>
<tr>
<td><strong>System Settings</strong></td>
<td></td>
</tr>
<tr>
<td>Use mass flow</td>
<td>Yes or No depending on preference. Mass flow is easier to balance.</td>
</tr>
</tbody>
</table>

**Important note on Ventsim releases** – the gas density calculation algorithms in Ventsim for high pressure conditions were updated for revision 5.1.2.2 or later. Earlier revisions of Ventsim may fail to simulate high pressure systems described in this paper.
ESTABLISHING A GAS RETICULATION MODEL

In both negative pressure gas drainage or positive pressure nitrogen injection systems it is necessary to fix flow intake or outlet rates as they will be dictated either by gas emission rates in gas drainage systems or valve settings in nitrogen injection systems. Furthermore, gas composition at intake points (boreholes or nitrogen plant outlet) must be pre-set. The recommended strategy for establishing a gas reticulation model is as follows (this creates a separate system unlike Ventsim’s duct builder tool):

1. All reticulation system entry or exit points must be connected to the surface or alternatively, for nitrogen injection, the pipe outlet can be connected to the ventilation model. This provides a means of sizing pipes with or without the complexity of connecting to the ventilation part of the model.

2. All entry or exit points, except two, must be fixed flow or have an equivalent fan curve applied to them. It is convenient to use mass flow for this balance as values are independent of temperature or pressure.

3. There must be one entry or exit point to the reticulation system that is not a fixed or a fan/pump branch. This avoids potential conflicts in the Ventsim mesh selection routine. The easiest strategy is to use one very high resistance branch that allows free mesh selection but does not lead to errors or high leakage rates.

4. For gas drainage systems, the inbye pipe range pressure will be that applied to hole collars. This is set by having a single inbye input branch with a specified fixed pressure drop, for example -5.0 kPa for collar pressures of that value. For nitrogen injection systems, the pipe pressure is set by an inbye pressure loss representing a control valve(s).

5. All entry points, that are not fresh air, must have the gas composition specified. Entry points to gas drainage systems that are leakage paths with fresh air will result in dilution of the gas mixture in the pipe. It is therefore necessary to run an “air” then “gas” then “air” simulation sequence to obtain the correct result.

6. It is recommended that airway type and airway friction factor pre-sets are established so that changes to the whole circuit can be made by changing a single value. This is useful when attempting to match the model to observed values. Unless survey results are very detailed, this is all that will be available in any event.

To set a model up without knowing actual losses in the system, the suggested range of friction factors for “clean” pipes is shown in Table 2 and shock losses for pipe fittings in Table 3. Actual pipe resistances will then be very much dependent on the degree of obstruction to due accumulation of water or drill fines.

Table 2: Friction factors for clean pipes

<table>
<thead>
<tr>
<th>Diam mm</th>
<th>0.01</th>
<th>0.1</th>
<th>0.5</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poly pipe</td>
<td>k N/m^4</td>
<td>k N/m^4</td>
<td>k N/m^4</td>
<td>k N/m^4</td>
<td>k N/m^4</td>
<td>k N/m^4</td>
</tr>
<tr>
<td>Galvanised pipe (welded)</td>
<td>0.0018</td>
<td>0.0029</td>
<td>0.0046</td>
<td>0.0057</td>
<td>0.0065</td>
<td>0.0073</td>
</tr>
<tr>
<td>100</td>
<td>0.0018</td>
<td>0.0029</td>
<td>0.0046</td>
<td>0.0057</td>
<td>0.0065</td>
<td>0.0073</td>
</tr>
<tr>
<td>200</td>
<td>0.0014</td>
<td>0.0022</td>
<td>0.0031</td>
<td>0.0037</td>
<td>0.0042</td>
<td>0.0046</td>
</tr>
<tr>
<td>400</td>
<td>0.0013</td>
<td>0.0020</td>
<td>0.0028</td>
<td>0.0033</td>
<td>0.0037</td>
<td>0.0040</td>
</tr>
<tr>
<td>600</td>
<td>0.0013</td>
<td>0.0019</td>
<td>0.0026</td>
<td>0.0031</td>
<td>0.0035</td>
<td>0.0037</td>
</tr>
</tbody>
</table>
Table 3: Notional shock loss factors for pipe fittings

<table>
<thead>
<tr>
<th>Item</th>
<th>X</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tee, Flanged, Dividing Line Flow</td>
<td>0.2</td>
</tr>
<tr>
<td>Tee, Threaded, Dividing Line Flow</td>
<td>0.9</td>
</tr>
<tr>
<td>Tee, Flanged, Dividing Branched Flow</td>
<td>1</td>
</tr>
<tr>
<td>Tee, Threaded, Dividing Branch Flow</td>
<td>2</td>
</tr>
<tr>
<td>Union, Threaded</td>
<td>0.08</td>
</tr>
<tr>
<td>Elbow, Flanged Regular 90°</td>
<td>0.3</td>
</tr>
<tr>
<td>Elbow, Threaded Regular 90°</td>
<td>1.5</td>
</tr>
<tr>
<td>Elbow, Flanged Long Radius 90°</td>
<td>0.2</td>
</tr>
<tr>
<td>Elbow, Threaded Long Radius 90°</td>
<td>0.7</td>
</tr>
<tr>
<td>Elbow, Flanged Long Radius 45°</td>
<td>0.2</td>
</tr>
<tr>
<td>Return Bend, Flanged 180°</td>
<td>0.2</td>
</tr>
<tr>
<td>Return Bend, Threaded 180°</td>
<td>1.5</td>
</tr>
<tr>
<td>Globe Valve, Fully Open</td>
<td>10</td>
</tr>
<tr>
<td>Angle Valve, Fully Open</td>
<td>2</td>
</tr>
<tr>
<td>Gate Valve, Fully Open</td>
<td>0.2</td>
</tr>
<tr>
<td>Gate Valve, 1/4 Closed</td>
<td>0.3</td>
</tr>
<tr>
<td>Gate Valve, 1/2 Closed</td>
<td>2.1</td>
</tr>
<tr>
<td>Gate Valve, 3/4 Closed</td>
<td>17</td>
</tr>
<tr>
<td>Swing Check Valve, Forward Flow</td>
<td>2</td>
</tr>
<tr>
<td>Ball Valve, Fully Open</td>
<td>0.1</td>
</tr>
<tr>
<td>Ball Valve, 1/3 Closed</td>
<td>5.5</td>
</tr>
<tr>
<td>Ball Valve, 2/3 Closed</td>
<td>200</td>
</tr>
<tr>
<td>Diaphragm Valve, Open</td>
<td>2.3</td>
</tr>
<tr>
<td>Diaphragm Valve, Half Open</td>
<td>4.3</td>
</tr>
<tr>
<td>Diaphragm Valve, 1/4 Open</td>
<td>21</td>
</tr>
</tbody>
</table>


DISPLAYING PIPES

A problem arises when attempting to display small diameter pipes on a mine model involving normal airway dimensions and scale of the mine e.g. a 400 mm x 3.0 km long pipe will not display clearly while viewing an overall mine layout.

One solution to this problem is to create a shadow of the pipe system as follows, Figure 1:

1. Create an “Air Type” called “pipe shadow” or similar.
2. Select a colour similar to the pipes but with some degree of contrast.
3. Duplicate then group then exclude the entire reticulation system.
4. Select the excluded group and make some visible size (e.g. 2.0 m wide x 0.5 m high) then offset 0.5 m below the actual reticulation system.

![Figure 1: Displaying pipes using excluded airway shadows](image)

The shadow "Air Type" can then be turned on or off as required and has no effect on the model.

DISPLAYING RETICULATION FITTINGS AND DEVICES

Pipe fittings (e.g. water traps or valves) can be shown by establishing a zero resistance pre-set resistance and assigning an icon. This is a particularly useful strategy for all Ventsim models to show where infrastructure is located, Figure 2. That is, the icon is shown but has no effect on simulation results.
The recommended method for establishing gas drainage system input points is to use multiple short fixed length airways, Figure 3 at each entry point. The first section is connected to the surface and has the gas composition specified. This provides the NTP entry density and gas properties at hole collars. The second is the fixed flow rate or the inbye fixed pressure drop which sets the individual hole flow rate or collar pressure. The third provides a monitoring point with a single value; otherwise the reported data value will be the branch average.

WT-Water trap, V-Valve, FA-Flame arrestor, P-Pumps

The mass balance for the system is then determined by the sum of fixed hole collar flow rates which will be determined by that of the total reporting to the surface pump station minus that through leakage paths or the single high resistance branch for mesh selection, Table 4.

Table 3: Mass Balance and icons for a gas drainage system

<table>
<thead>
<tr>
<th>Gas Drainage Feature</th>
<th>Mass Flow kg/s</th>
<th>Number in Circuit</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fix flow (1)</td>
<td>mc</td>
<td>Nc</td>
<td>One for each collar or set of holes</td>
</tr>
<tr>
<td>Fix DP</td>
<td>mdp</td>
<td>1</td>
<td>Fix collar DP e.g. -5,000 Pa</td>
</tr>
<tr>
<td>High res</td>
<td>mr</td>
<td>1</td>
<td>Open split (Q = (DP/R)^n) very low flow</td>
</tr>
<tr>
<td>Fix flow (2)</td>
<td>mp</td>
<td>1</td>
<td>Pump flow (mp = mr + mdp + mc \times Nc)</td>
</tr>
</tbody>
</table>

For nitrogen injection systems, the recommended strategy is to set an NTP flow or mass flow rate (at a set composition) into the plant from surface pressure then set the back pressure by applying fixed pressure drops to the end of pipes where control valves will, in reality, be located, as shown in Figure 4. Again, a single very high resistance branch is used to provide a theoretical open split for mesh selection.
This model shows nitrogen pipes discharging to atmosphere for the purpose of sizing pipes. However, they can also be connected to the mine’s ventilation model to predict inertisation times.

The mass balance for the system is then determined by the sum of fixed hole collar flow rates which will be determined by that of the total flow rate provided by the nitrogen plant distributed to outlet points, Table 4.

<table>
<thead>
<tr>
<th>Feature</th>
<th>Mass Flow Kg/s</th>
<th>Number in Circuit</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fix flow (1)</td>
<td>mc</td>
<td>Nc</td>
<td>One for each injection point except one</td>
</tr>
<tr>
<td>Open split</td>
<td>mo</td>
<td>1</td>
<td>Open split on low pressure side</td>
</tr>
<tr>
<td>Fix DP</td>
<td>mdp</td>
<td>1</td>
<td>Fix back pressure on reticulatin system</td>
</tr>
<tr>
<td>High res</td>
<td>mr</td>
<td>1</td>
<td>Open split (Q = (DP/R)^n) very low flow</td>
</tr>
<tr>
<td>Fix flow (2)</td>
<td>mp</td>
<td>1</td>
<td>N2 plant flow mp = mr + mo + mi x Ni</td>
</tr>
</tbody>
</table>

**APPLICATION TO A GAS DRAINAGE SYSTEM**

The modelling strategy for gas drainage systems described above has been applied to a mine’s gas drainage system for which flow rates, gas composition and pipe pressure differentials were measured. The purpose of the analysis was to assess the need to increase the pipe diameter in the reticulation system for inbye extension.

The geometry and characteristics of the current gas reticulation system together with the Ventsim model results are shown in Figure 5. The main issues were as follows;

1. At a flow rate of about 1.5 m³/s NTP at 85% CH₄, the inbye mains pipe had a pressure differential of about -4.1 kPa and the pump inlet pressure was -15 kPa.

2. An allowance of 20 kPa back pressure on the pump outlet was also made to allow for delivery of gas to gas utilisation and methane destruction infrastructure.

3. The existing mains 450 mm pipe was at about 5.0 km from the pump station borehole. The plan was to extend this by up to 6.0 km inbye for a potential life of mine (LoM) length of 11 km.

4. The success of pre and post drainage strategies meant that the plan flow rate for future gas drainage requirements was to be increased to 3.0m³/s NTP mixed 85% CH₄ at up to -30 kPa pump inlet pressure.

5. Recognising that the 450 mm pipe could not meet LoM gas drainage requirements, the main question to be addressed was what pipe size would be appropriate?
Figure 5: Ventsim model of existing gas reticulation system
A survey of all flow rates and pressures for individual drilling stubs was undertaken by the mine to determine the distribution of gas flow rates and pressure losses in the system. For example, the flow rate and pressure differential profile of MGxx is shown in Figure 6. This type of information is invaluable for tuning any type of Ventsim model.

Once the reticulation system was set up in Ventsim with the gas flow balance described by the survey, the pre-set pipe resistance was modified to match observed pressure differentials. This strategy determined an effective friction factor for installed pipes (including fill and valves) of 0.0055 Ns²/m⁴ or about 20% higher than for a clean pipe. Of course, any significant accumulation of water or drilling fines in a single pipe could completely change the result.

**DESIGN OUTCOMES**

The tuned model was then extended to 11 km and the flow rate increased to 3.0 m³/s NTP 85% CH₄ at the inbye gates. The model was then run using pre-set “Air Types” for pipe diameters of 600 mm to 900 mm but with the pre-set pipe friction factor set to that obtained by matching observed flow and pressure data. The outcome, for a base case of 3.0 m³/s NTP at 85% CH₄ and 4.2 m³/s NTP at 60% CH₄, was that a 750 mm pipe would be required for extension to 11 km, Figure 7. This would include replacing the existing 450 mm diameter pipe with a pipe of similar diameter. The mine has now commenced with installation of 750 mm pipes.
For general design of gas drainage systems operating at low pressures (50 to 70kPa (abs)), this analysis provided a limiting pipe velocity of about 14 m/s. For example, and in round numbers, 3.0 m³/s NTP would be about 6.0 m³/s at 50 kPa (abs) which would require a 738 mm pipe for 14 m/s at the pump inlet.

APPLICATION TO PIKE RIVER RE ENTRY NITROGEN INJECTION SYSTEM

Important qualification – the Pike River Recovery Agency (PRRA) has given permission to use the following information for this paper under its policy of openness and transparency. This information, analysis and design outcomes are preliminary, subject to further work and risk assessment. Not all, if any, of this work will necessarily be employed in the re entry process.

Following a methane explosion in 2010 in which 29 miners were killed, Pike River coal mine is currently sealed. Other than some variations adjacent to the portal seal during periods of rising barometric pressure, the mine’s atmosphere is now more than 90% methane. As part of the proposed re-entry of the drift, to be undertaken sometime in 2019 for forensic examination purposes (PRRA, 2018), the plan is to initially displace methane with nitrogen then maintain nitrogen injection to the body of the mine while the drift is recovered in an atmosphere compliant with explosion risk zone 1 (ERZ1) standards and as shown in Figure 8.

![Figure 8: Pike River Mine Re-entry nitrogen reticulation system and surface topography](image)

The nitrogen reticulation system, Figure 8, comprises the following:

1. A membrane nitrogen plant delivering up to 420 l/s NTP at 98%N₂ at up to 10bar (point A). There are also three back up cryogenic units.
2. Galvanised steel pipe (110 mm to 305 mm ID) from amenities to the portal area (points A – B – C). This provides for nitrogen injection to flush the drift.

3. Approximately 4.1 km of twin 90 mm (76 mm ID) hoses to the injection borehole collars at an elevation approximately 400 m above the portal, points D to E. Due to the steep terrain, these will be installed by ground crew after being dropped in by helicopter. It is therefore essential that they be sized appropriately.

4. Two injection boreholes (141 m x 77 mm ID and 98 m x 150 mm ID) into the coal workings. These provide for nitrogen injection into the coal mine workings to maintain an inert atmosphere during re-entry to the drift.

The purpose of the modelling exercise was as follows;

1. To demonstrate that two rather than one 4.1 km hose was required to connect the plant to holes into the workings.

2. To estimate what the purge times would be (using dynamic gas simulation).

3. To provide a graphical representation for explanation of the injection strategy to other interested parties.

The model employed included a normal 3D representation of the mine to scale combined with a schematic representation of the nitrogen reticulation system, Figure 9. In this case, the exit points from the injection holes were connected to the mine’s ventilation model using dummy low resistance pipes. A spreadsheet approach was also employed to check outcomes of this analysis.
Figure 9: Combination of mine scale and nitrogen system schematic in a single ventsim model
A point to note when modelling gas drainage or nitrogen injection systems is that the characteristics of flame arrestors may not follow the square law. This could be significant when considering flow rates through venturi assemblies without applying venturi pressure i.e. under natural buoyancy.

The previously used Pike River venturi assembly, Figure 10, was found to follow a $P \propto Q^{1.8}$ relationship with new 150mm flame arrestors following a $P \propto Q^{1.5}$. This is due to transitional flow regimes through the flame arrestor packing. This relationship can be included in the Ventsim presets.

![Figure 10: Venturi assembly and flame arrestor characteristics](image)

**DESIGN OUTCOMES**

This model, together with crosschecks with spreadsheet calculations, provided the following design outcomes:

1. Two 4.1 km 90 mm nitrogen pipes will be required to deliver full plant capacity to the mine working’s injection boreholes at less than 4 bar (400 kPa). This analysis also provided a planning value of approximately 300 l/s NTP through a single pipe should the other fail.

2. Two additional 150 mm boreholes will be required, one for increased injection capacity to the mine workings and another for increased exhaust capacity.

3. Drift purge times could vary between 2 and 7 days depending on the status of additional holes and degree of connectivity between the drift and workings through a fall of ground at the top of the drift.

Currently, the model is conceptual and subject to tuning to observed conditions as the re-entry process proceeds. In this respect, nitrogen injection to the portal commenced on 12th December 2018 and, as at 14th December 2018, successfully purged the drift (<3.0% CH₄ and <2% O₂) so far as the 140 m point in preparation for the first phase of re-entry.
CONCLUSIONS

This work demonstrates that Ventsim can be used to model gas reticulation systems at pressures significantly higher and lower than normal mine ventilation systems. Results are consistent with those from spreadsheets based on conventional, and well tested, calculation methods.

Application of Ventsim software to gas reticulation systems aids in decision making with respect to pipe sizing and pressure control. It also provides a means of documenting the location and status of gas drainage infrastructure for ongoing system management.

Unlike pure ventilation models, in which ventilation flow rates are determined by fan characteristics and regulator settings, it is necessary to pre-determine gas flow rates and take a “fixed flow” approach to modelling of both gas drainage and nitrogen reticulation systems i.e. assume that valves will be altered as or when required to modify gas flow rates. The main outcomes for design purposes are the resultant pipe losses and operating pressures being dependent on pipe diameters employed together with location and settings of valves.

A further use for Ventsim models is the application of pictures to fixed resistance icons so that infrastructure can be located on the plans or model without necessarily being used for calculation purposes. This provides a means of planning and communicating infrastructure locations.

REFERENCES


WHAT ARE THE OUTCOMES FROM 21 YEARS OF LEVEL 1 EMERGENCY EXERCISES IN QUEENSLAND?

Martin Watkinson¹

ABSTRACT: Level 1 emergency exercises have been run annually in Queensland since 1998. The 21st exercise was run in July 2018 at Grosvenor mine. This paper will discuss and identify the improvements to emergency response and coal mine safety that have resulted from the running of these exercises.

INTRODUCTION

The Wardens inquiry into the explosion at the Moura No. 2 Mine in August 1994 recommended -“Emergency procedures should be exercised at each mine on a systematic basis, the minimum requirement being on an annual basis for each mine.” (Windridge, et al., 1996). In December 1996 the “Approved Standard for the Conduct of Emergency Procedures Exercises” was published and was subsequently reissued as Recognised Standard 08 (RS08) Conduct of Mine Emergency Exercises. This document provides guidelines for conducting mine site emergency exercises as well as the requirement for a test of state-wide emergency response by holding a Level 1 Mine Emergency Exercise at one mine on an annual basis. Since 1998 there have been 21 Level 1 Mine Emergency Exercises held in Queensland, Australia.

Several papers have been written primarily on individual exercises or summarising the exercise status to date. There have been 863 recommendations made as a result of the 21 exercises and a summary spreadsheet has been prepared to collate the recommendations. One paper in particular Watkinson (2006) provides an update to industry as a result of the exercises and noted that exercise reports were not freely available. In particular, one coal mine that held the Level 1 exercise had 20 hard copies of the report sent to the mine. A couple of years later the mine was sold and not one copy of the report was available. Copies of all 21 reports are now available on the Queensland Government website.

SCENARIOS

There have been several scenarios used over the 21 exercises and all have been used more than once. Scenarios are developed after visiting the mine and reviewing the mine site hazards. Sometimes scenarios are developed to reinforce an issue that industry is facing. Most of the scenarios will require coal mine workers (CMWs) to wear breathing apparatus, Self-Contained Self-Rescuers (SCSR) and/or Compressed Air Breathing Apparatus (CABA) as part of their self-escape. This normally also involves changing over from one SCSR to a new one or from SCSR to CABA.

RS08 also requires a deployment of Queensland Mines Rescue Service (QMRS) underground (UG) to effect a rescue or recovery.

In order to create a situation where CMWs have to effect an escape using breathing apparatus there are a limited number of scenarios available. Likewise the need to deploy QMRS would

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require an irrespirable atmosphere to be present otherwise the mine could undertake recovery operations using mine staff. A list of the scenarios is given in Table 1.

Table 1 List of Scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Mine and date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface fire</td>
<td>Newlands 2008</td>
</tr>
<tr>
<td></td>
<td>Ensham 2013</td>
</tr>
<tr>
<td></td>
<td>Broadmeadow 2017</td>
</tr>
<tr>
<td>Roof fall</td>
<td>Kestrel 2001</td>
</tr>
<tr>
<td></td>
<td>Aquila Mine 2011</td>
</tr>
<tr>
<td>Explosion</td>
<td>Southern Colliery 1998 (FI)</td>
</tr>
<tr>
<td></td>
<td>Oaky No 1 2004 (FI)</td>
</tr>
<tr>
<td></td>
<td>Grasstree 2007 (FI)</td>
</tr>
<tr>
<td></td>
<td>Oaky North Mine 2012 (FI)</td>
</tr>
<tr>
<td></td>
<td>North Goonyella 2015</td>
</tr>
<tr>
<td></td>
<td>Grosvenor 2018 (FI)</td>
</tr>
<tr>
<td>Conveyor fire</td>
<td>Kenmare 1999</td>
</tr>
<tr>
<td></td>
<td>Cook Colliery 2009</td>
</tr>
<tr>
<td></td>
<td>Kestrel 2014</td>
</tr>
<tr>
<td></td>
<td>Grasstree 2016</td>
</tr>
<tr>
<td>Other UG fire</td>
<td>Newlands 2000</td>
</tr>
<tr>
<td></td>
<td>North Goonyella 2002</td>
</tr>
<tr>
<td></td>
<td>Crinum 2003</td>
</tr>
<tr>
<td></td>
<td>Moranbah North 2005</td>
</tr>
<tr>
<td></td>
<td>Broadmeadow 2006</td>
</tr>
<tr>
<td></td>
<td>Carborough Downs 2010</td>
</tr>
<tr>
<td>Spontaneous combustion</td>
<td>Part of North Goonyella 2002</td>
</tr>
</tbody>
</table>

*FI is frictional ignition

MAJOR RECOMMENDATION

Surface

Many of the recommendations in the early exercises related to the organisation of the emergency response in particular the formation and running of the incident management team. The handling of data, debrief information and the use and number of duty cards also fall into this category.

Duty cards were often not suitable for the task to be carried out by the card holder. The cards then developed into task sheets sometimes several pages long and tasks critical to the situation were sometimes on the second page of the document and not acted upon in a timely manner.

There were also observations on the ability of the control room operators to quickly interpret the gas data and a lack of awareness of the range of gas instruments.

Debrief of CMWs and the care of injured personnel was also raised as areas for improvement.

QMRS also came under criticism for the approach used in assessing the hazards associates with underground deployment in particular the QMRS deployment guidelines.

Underground

In all the exercises the ability of CMWs to don a SCSR and change over either to another SCSR or CABA is tested. Video footage is taken of this process to enable a thorough assessment of the issues associated with this activity. There have been many cases where CMWs have failed to undertake this process successfully. Every exercise report makes recommendations to improve this outcome. The main emphasis is on the training of the CMWs and the fact that this is an area where CMWs are responsible for their own safety.
Low light evacuation is also tested using smoke goggles as is the ability of CMWs to communicate to the surface when wearing a SCSR using non-verbal communications. There was one mine where two different forms of non-verbal communication were used and many recommendations have been made to standardise the system across Queensland. This was fortunately captured by the video recording and is a timely reminder for the need to standardise across industry. One positive outcome from this was the ability of the control room operator to quickly identify the issue and make positive communications with the escaping CMWs.

Lifelines have been introduced into many mines and the lack of maintenance of the system has often led to issues. Again recommendations have been made on standardising the configuration of cones and markers on the lifelines.

The CMWs who safely evacuated form the Moura No 2 explosion did so by using UG vehicles. There have been a number of occasions in exercises where the evacuating CMWs did not try to use a vehicle that was available.

Mine site specific recommendations are made where there are issues with particular evacuation routes i.e. flooded walkways, lifelines obstructed or air door pressures. Whilst these are specific to the mine site all coal mines Site Senior Executives (SSEs) should organise a review of their mine as a result of the Level 1 exercise report including mine specific issues.

**OUTCOMES**

Over the 21 years there have been distinct improvements made both underground and on the surface. Every year recommendations are made that identify opportunities for improvement. The Level 1 exercise drives a continuous improvement process and these outcomes are evidence of some of the improvements that have occurred.

One improvement has been the adoption of the Australasian Inter-service Incident Management System (AIIMS 2017) as method of coordinating the surface response to the presented scenario. The QMRS Mine Emergency Management System (MEMS) for emergency response is based on AIIMS and was developed to tailor the approach to coal mine specific scenarios. Many mines send their management teams to the MEMS course in the months leading up to the exercise. This system utilises an incident management team which has three working teams to support the decision making process these are operations, planning and logistics. The exercise team invited the senior sergeant from the Emerald Police to review the surface emergency response at the kestrel exercise in 2014. He was positive in that what he saw in the system was being used and recognised the process; the one thing he noticed was there did not appear to be an intelligence cell. He noted that the control room appeared an ideal location for this function.

In the past three years there has been an adoption of electronic data management systems, which assist in the processing of the data and makes the data available across all the sections of the surface response. Previous exercises have recommended the adoption of such a system.

Each mine uses this opportunity to review the site emergency response plan and update it in the months before the exercise. Many conduct desk top exercises and mock evacuations in the months before the exercise.

Underground there have been many improvements as a result of the focus on emergency response and Level 1 exercises. Many mines now have:

- Wind chimes / audible sounds to assist in finding caches, these are sometimes also attached to the life line should the ventilation be impaired.
• Signage to designate the primary and secondary escape routes, unfortunately this is not standard across Queensland mines.

• The emergency number 555 has been adopted at all but one Queensland mine. Many mines in NSW also use this number. The one mine in Queensland that does not have 555 as the emergency number due to reported technical issues with their telephone exchange.

• Blind men’s sticks or candy canes to assist CMWs to evacuate in low light conditions.

The use of Change Over Bays (COBs) was recommended in the first Level 1 exercise at Southern Colliery. At that stage all mines in Queensland other than one (Newlands) had an escape strategy based on using SCSRs and caches for escape. In 2007 the exercise was held at Grasstree mine which is connected to the old Southern Colliery workings again recommended the use of COBs. The use of COBs was evaluated as part of the 2016 Level 1 exercise at Grasstree.

There is now a wider adoption of CABA for self-escape; this was tried and demonstrated during the Newlands exercise in 2000.

The Chief Inspector of Coal Mines brought together groups of industry personnel to develop a draft recognised standard on emergency management as a result a review of the recommendations from the Task Group 4 Committee’s report. Task Group 4 was formed as a result of the recommendations from the Moura No 2 Wardens Inquiry report. This standard covers the first four hours of the mine site response. One of the information sources for this report was the summary spreadsheet of all the level 1 exercise recommendations. One part of this is the standardisation of:

• Escape way signage, markings and the colour coding of the mine escape plan to match.

• Hard hat colours at coal mines to identify trainee miners, inexperienced miners, statutory officials and mines rescue trained personnel

• Standard for non-verbal communication

• Standard for lifelines

• Standard for equipment at caches. Each mine to ensure that all caches are established to a mine site standard ie the same distance into the cut though, location of telephone/communications, first aid equipment

Coal mines are now replacing their older SCSRs and the older units have been available for CMWs to wear during the exercise. CABA is readily available and each mine allows at least one team to evacuate using their CABA suits. Where CMWs are wearing SCSR and are underground awaiting their recover by QMRS it is an ideal opportunity to test the longevity of the SCSR at rest. The CMWs are asked if they are willing to undertake this test and many including CMWs new to the industry have participated in this test. Some impressive results have been obtained as a result of this testing 2 hours 25 minutes at Grasstree (25 minutes walking and 2 hours at rest) and 3 hours 18 minutes from a CSE 100 at Grosvenor whilst at rest.

QMRS had an inflexible deployment guideline which underwent a risk based review in conjunction with New South Wales Mine Rescue Service (NSWMRS) to develop the Mine Re-entry Assessment System (MRAS). MRAS is designed to facilitate the re-entry of mines rescue
teams underground utilising mine site gas data and information to enable the Incident Management Team (IMT) to make sound decisions on the safety of mines rescue teams entering or re-entering a mine having CMWs remain underground.

Social media interaction has been tested at a number of the exercises. This was done by creating fake social media pages and emailing them to the mining company personnel for a response. There are differing opinions on the outcomes with some mining companies not officially engaging in social media. Whilst there is no direct outcome from this interaction it has highlighted the issue will need to be addressed during an emergency event.

**CONCLUSIONS**

Emergency response is not an everyday process for coal mine management to follow and there are often issues relating to the incident management in particular where electronic database systems are being used as they are not utilised to their full capacity. Mines should be using these systems as a part of their everyday management of the mine so that this just becomes another familiar tool to use during the emergency response process.

Another issue faced is the availability of personnel to fill the duty cards. One mine had 93 duty cards. That is probably more duty cards than there will be personnel on site during the proposed 2019 exercise. Whilst these are scalable systems it must be remembered that this system must be simple and easy to follow.

One exercise observer once posed the question should duty cards be written as a desired outcome rather than a process or number of dot points?

Whilst the exercise team always informs the host mine of the exercise intent and that it is a test of the mine and state response it is hard to break the opinion that this is pass or fail test. Comments such as you say that but don’t mean it are common from some management staff. The best reply is it’s like playing golf you are playing against yourself and trying to improve. Embrace it and enjoy the experience. It is for this reason that exercises are sometimes run on nightshift or weekend when senior management are not present onsite. This is even more important as some mines most of the senior staff operates on a Fly in Fly out (FIFO) basis. This makes the use of electronic database systems even more important as technical support to the exercise response can be provided remotely.

Social media will be a major issue in a real event. An example of this was the social media posts of the smoke coming out of the shaft at North Goonyella coal mine in September 2018. Each mine and mining company will need to address their response to social media posts. A proactive approach is best. It would be useful to have pre-prepared background information available on the mine including photographs and video footage. Factual information then needs to be released when possible as is the case for media releases. During a major incident there will be intensive press coverage and being prepared is the best position to be in.

Several exercises have involved Queensland Police deployment to site. If the mine has a fatality or multiple fatalities this could vary the Police response at the mine. The Police could take control of the response if they were not convinced of the diligence of the mine site response using the MEMS/AIIMS (2017) process and following it correctly. The Police would most likely utilise information and advice from the Queensland mines inspectorate and will defer control to them as they have the technical expertise. Where there are fatalities the Police are the coroners’ representative and have ultimate control.

Any mine that has an emergency escape system based on SCSRs should have a self-escape process based on the use of COBs.
There are a large number of contract CMWs who can work at a different mine every week and therefore mines must adopt the standard practices that are being developed in the Recognised Standard for Emergency Management. It was identified at the 2015 North Goonyella exercise that two teams of evacuating CMWs used different protocols for non-verbal communications. CMWs at mine sites now embrace the Level 1 exercise and enjoy the opportunity to test their self-escape capability and are more than willing to participate in testing the longevity of SCSRs in a real underground test. Video footage of the 2018 exercise was very useful in demonstrating how things have improved with CMWs tapping stuck SCSRs to release them, assisting other CMWs who were struggling to don their SCSR and working as a team to self-escape. It was evident that the CMWs were well rehearsed and practiced in this process.

CMWs who are studying for statutory certificates are participating as assessors and report that they thoroughly enjoy the opportunity and experience offered by being involved. This involvement not only allows them to participate in the planning of the exercise but to build relationships with the inspectorate and industry safety and health representatives (ISHR) as well as other senior mining managers. These CMWs bring practical hands on approach to the running exercise and a realistic review of the underground response. Their involvement also means the learnings are quickly communicate to their mine sites and the crews that they work with.

It must be remembered that Queensland Level 1 Exercises are a learning opportunity for the mines and state services to test their response, communication systems and interactions with the aim of continual improvement of the whole state response system and it can be concluded that:

- Mineworkers are now more familiar with SCSR and CABA (ongoing refresher training still needed).
- MEMS was developed by QMRS and has been adopted by all of the mines. (More practice is still required). Coal mines must stick to the process.
- Any process that is used for emergency response must be practiced.
- Mine management needs to welcome the opportunity to test their emergency response plan and no longer regard it as a pass or fail test. It is the ideal opportunity to identify improvement opportunities.
- Industry needs to work on ways of sharing the lessons with each other as mutual safety is in everyone’s interest.
- Standardisation is critical with the numbers of contract CMWs currently employed in the industry.
REFERENCES


RELIABLE REFERENCING FOR FIXED AND MACHINE MOUNTED GAS DETECTORS

Ian Webster1, Thomas Steigler2

ABSTRACT: Diffusion type gas detectors are used in fixed, machine mounted and handheld applications in underground mines for the real time detection of flammable and toxic gases. The calibration of a gas detector – typically by a ‘bump’ or ‘challenge’ test – sets the reference against which all subsequent measurements are taken. Failure to properly calibrate a detector will introduce errors into every subsequent measurement, regardless of the claimed accuracy of the instrument. The vulnerability of the calibration process to poor calibration mask design and to ambient air velocities has been established. The real problem, however, is knowing if and how the calibration process has been compromised. This paper discusses recent experiments showing the susceptibility of gas detection and calibration to poor calibration mask design and to ambient air velocities, and outline new research utilising a computational fluid dynamics approach to analysing calibration mask performance.

INTRODUCTION

Fixed and handheld diffusion type gas detectors are used to sample ambient atmospheres for the presence of flammable and/or toxic target gases. The performance of such detectors is characterized by standards such as AS/NZS 60079-29-1 (for flammables) and AS/NZS 4641 (for toxics and oxygen). Compliance to these standards is generally not mandatory. However, for heavily regulated applications (such as underground coalmines) the need for compliance to performance standards is typically enforced through regulation.

Both AS/NZS 60079-29-1 and AS/NZS 4641 prescribed accuracy and response time requirements under nominated test conditions. Original Equipment Manufacturers (OEMs) will typically submit new detectors for testing, with fresh factory calibrations and free from any contaminations or occlusions, thus obtaining the best possible performance results from the nominated and accredited test laboratory. End users, in turn, are often reliant on such test results to help select detectors for a given application. Comparisons of sensor accuracy and response time, environmental immunities and, of course, cost will all impact on decision criteria.

FIELD APPLICATION

In practice, the accuracy and response times of diffusion type detectors will be compromised through various permutations of:

- Drift of zero point due to ambient conditions (e.g. temperature, humidity, atmospheric pressure) affecting sensing elements
- Drift of zero point due to ambient conditions affecting embedded electronics
- Variation (reduction) of sensitivity of sensing elements due to ageing
- Variation of sensitivity of sensing elements due to contamination
- Variation of sensitivity of sensing elements due to partial blockages
- Variation of sensitivity of embedded electronics due to ageing

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2 Graduate Engineer, Ampcontrol. Email: Thomas.steigler@ampcontrolgroup.com, Tel: +61 2 4961 9000
• Variation of sensitivity due to ambient conditions

OEMs will usually provide guidance on the use and maintenance of such detectors to:

• Minimize the likelihood of the above factors affecting detector performance (preventative actions)
• Rectify the sources of performance deterioration (e.g. cleaning filters)
• Compensate for the performance deterioration through field calibration

Such guidance might range from rudimentary inspections and cleaning of filter elements, through to testing and correction of detector zero and span readings under field calibration.

The vulnerability of field calibration

The absolute accuracy of a detector, as measured by an accredited laboratory, is validated by traceable calibrations of test equipment and the demonstrable competency of laboratory staff, often under a quality scheme such as The National Association of Testing Authorities (NATA).

By comparison, the absolute accuracy of a detector calibrated in the field is totally reliant on both the competency of the operator, as well as the efficacy of the apparatus used to conduct the calibration. Experiments have shown that a number of factors can adversely affect field calibrations:

• Poor design of calibration masks (calibration cups)
• Poor fitment of calibration masks
• Incorrect flow rate of test gas
• Pressurisation of gas under the calibration mask (often by excess flow rate)
• Ambient air velocities diluting test gas concentrations
• Ambient air velocities over-exciting (catalytic) sensors

It is then quite possible that gas detectors in the field are routinely failing to perform to advertised levels of accuracy.

Poor design of calibration masks

Gas detector OEMs do not always apply the same degree of performance validation to calibration mask design compared with the detectors themselves.

Calibration masks are generally, but lamentably, disregarded as critical components in the gas detection safety system. Indeed, numerous anecdotes circulate describing the deployment of rags, rubber gloves, disposable drinking cups and other paraphernalia as makeshift calibration masks.

The principal function of a calibration mask (or cal cup) is to uniformly deliver a test gas to the sensing element, without imposing any physical or chemical disturbances that might introduce errors in the gas reading. Such disturbances might include contamination (such as silicones from tubing or rubber gloves), residual ‘sticky’ gases in the tubing or calibration masks, pressurisation of the gas at the sensor (see below), or temperature differences between test gas and ambient air.

The ideal calibration mask will deliver test gas to the sensing element at ambient temperature and pressure, in a homogeneous field at minimal velocity. (Some velocity is inevitable in order to introduce the test gas to the void.)
Factors to consider in the design of a ‘good’ calibration mask include:

- **Fitment of the seal between the calibration mask and the gas detector:** studies have shown that while sealing is not critical in laboratory environments with relatively still air, the impact of ambient air velocities typically found in underground mines will greatly inhibit the efficacy of the ensuing calibration.

- **Design to accommodate all arrangements of the detector and its environmental housings:** some OEMs provide calibration masks that require various ingress protection elements to be removed prior to calibration. The removal of these elements invariably changes the gas flows in the detector, resulting in optimisation in one configuration, but reduction in another.

- **Management of flow rate:** OEMs typically nominate a given flow rate for the test gas at which the calibration is to be undertaken. However, the mask (and procedure) should be designed with a significant robustness to varying flow rates caused by operator error of incorrect parts (such as flow rate regulators).

- **Management of laminar and turbulent flows inside the mask:** the shape, size and location of various orifices inside the mask can dictate whether the resulting gas flows are laminar or turbulent. Turbulent flows are desirable from the point of gas mixing; laminar flows can result in non-uniform gas concentrations inside the calibration mask.

- **Locations of inlet and exhaust ports:** the respective ports should be spatially separated to avoid ‘short-circuiting’ the gas flows directly from inlet to exhaust. The ports should be aligned to enhance gas mixing and uniformity inside the mask.

- **The material(s) used the construction of the mask and connecting hoses should not react chemically with either test gases or with the sensing technologies deployed in the detector.** It is argued that performance validation of gas detectors should also mandate performance testing of OEM calibration masks and operating instructions to ensure end-users are provided with a demonstrably reliable calibration procedure.

**Poor fitment of calibration masks**

It is known that external air flow can impact the calibration process. Figure 1 shows three response curves for a single gas detector, using an identical test gas, but with different calibration masks and degrees of sealing.

A test gas of 2.43 % methane in air was delivered to a given detector via a calibration mask with a flow rate of 0.5 L/min. The reading was allowed to reach steady state before a 12 V fan was introduced to simulate external airflow past the calibration mask. The fan was placed at a distance of 200 mm behind the mask outlet, then 200 mm away at a 90° angle to the outlet, and then 200 mm away pointing directly at, and in line with the outlet. The fan was held at each location for 30 s. Finally, the fan was placed directly at the outlet.

The trace labelled ‘V1’ is a calibration mask designed for the given detector, but with a poor mating seal between mask and detector leaving a gap of 1 mm. The impact of the moving air from the fan on measured gas concentration is substantial.

The trace labelled ’V1 - Sealed’ is the same calibration mask, but with the previous gap sealed to prevent leakage between the mask and the detector. The improvement in measurement consistency is quite evident, even when subjected to the same series of imposed fan disturbances.

The trace labelled ‘V2’ represents an improved calibration mask design, with even greater immunity to fan (moving air disturbances). See Figure 2.
In each case, the calibration masks had an exposed exhaust port area equivalent to the inlet port area.

![Figure 1: Sensitivity of methane detector to ambient air velocity. V1 has poor seal between calibration mask and gas detector exposing vulnerability to movement of surrounding air. (Steigler, 2019)](image)

Table 1 shows the results of testing of a number of gas detectors to sensitivity against incorrect flow rate of test gas.

In each instance, the detector was calibrated in accordance with OEM instructions. The flow rate was then varied around the OEM nominal specification.

As can be seen (and might be expected), the measured concentration of the gas varied with flow rate.

![Figure 2: Calibration mask ('V2') designed for a particular gas detector using computational fluid dynamic analysis methods to optimise the resulting mask performance. (Steigler, 2019)](image)
Table 1: Sensitivity of a sample of gas detectors to varying flow rate of test gas. (Bale, 2013)

*Note: Detector was calibrated at 2.50% with a 0.25lpm flow rate

<table>
<thead>
<tr>
<th>Flow Rate (L/min)</th>
<th>Adapter Setting</th>
<th>Pressure (Pa)</th>
<th>Reading @ 2.50 (%Vol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>Manufacturer’s recommendation</td>
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</tr>
<tr>
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<td>Manufacturer’s recommendation</td>
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<td>3.0</td>
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*Note: Detector was calibrated at 2.50% with a 0.86lpm flow rate

<table>
<thead>
<tr>
<th>Flow Rate (L/min)</th>
<th>Adapter Setting</th>
<th>Pressure (Pa)</th>
<th>Reading @ 2.50 (%Vol)</th>
</tr>
</thead>
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<tr>
<td>0.50</td>
<td>Manufacturer’s recommendation</td>
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<tr>
<td>1.00</td>
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<td>2.60</td>
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*Note: Detector was calibrated at 2.50% with a 0.25lpm flow rate

<table>
<thead>
<tr>
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<th>Adapter Setting</th>
<th>Pressure (Pa)</th>
<th>Reading @ 2.50 (%Vol)</th>
</tr>
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<tr>
<td>0.25</td>
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<tr>
<td>0.50</td>
<td>Manufacturer’s recommendation</td>
<td>10</td>
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<tr>
<td>1.00</td>
<td>Manufacturer’s recommendation</td>
<td>15</td>
<td>2.75</td>
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</table>

This sensitivity, however, need not be tolerated. Figure 3 shows the variation of measured test gas concentration against applied flow rate for the ‘V2’ mask. The exaggerated vertical scale magnifies the variability: 2.42% to 2.45% across a range of flow rates from 0.2 l/min to 1.0 l/min.

![Graph showing sensitivity of a superior gas detector to variations in test gas flow rate.](Steigler, 2019)

While the application of test gas to a detector under test should be within OEM specifications, the immunity to variations in flow rate can be designed with due diligence.

**Pressurisation of gas under the calibration mask**

The results in Table 1 show both the measured gas concentration and the pressure of the test gas measured under the calibration mask. It is observed that when the test gas flow rate is varied outside of OEM specification, the pressure under the mask increases.

The ideal gas law PV = nRT dictates that all other variables being equal, an increase in pressure (P) will result in an increase in measured gas concentration (n).
The pressure rise observed in Table 1, then, is the result of poor calibration mask exhaust porting, resulting in an accumulation of test gas under the fixed volume mask.

Empirical studies have indicated that a well-designed calibration mask should have an exhaust port of equivalent (or greater) cross-sectional area than the mask’s gas inlet. This ensures that pressure build-up under the mask is minimised.

Earlier versions\(^1\) of AS/NZS 60079-29-1 (2008) contained reference to an ideal pressure limitation of 50 Pa\(^2\) under the calibration mask. Limiting to this value ensured minimal pressurisation under the mask and so negligible effect on the measured gas concentration.

**Ambient air velocities diluting test gas concentrations**

The potential susceptibility of gas detectors and calibration masks to ambient air velocity was shown in Figure 1. In particular, when a gap existed between the calibration mask and the detector under test, the measured concentration was markedly reduced.

The reigning hypothesis to explain this observation is that the movement of the ambient air serves to evacuate the test gas from the calibration mask by venturi effect. In the experiment described, the gap was between the mask and detector body. Of course, other gaps, orifices or misalignments could also cause the same venturi effect.

It is relevant to recognise that this effect is difficult to observe in the field, where no frame of reference is available. Observation in a laboratory is easier, since invariably the ambient air is still, and can be forced by a controlled fan. It is crucial, then that the calibration process, equipment and technique be developed and validated under controlled conditions before application in the field (mine) where ambient conditions are essentially uncontrolled.

**Ambient air velocities over-exiting (catalytic) sensors**

In some instances, the introduction of moving air can apparently increase the measured concentration of gas – the reverse of the above venturi situation.

Catalytic methane detectors, unlike IR technologies, consume methane during the sensing process. Such detectors require a constant flow of methane, and oxygen, to sustain the oxidation process. Furthermore, the byproducts of the oxidation need to be exhausted from the bead surroundings.

In some situations the introduction of moving ambient air invigorates the oxidation process by one or more of:

- Forcibly introducing more methane to the catalytic bead
- Forcibly introducing more oxygen to aid the oxidation
- Evacuating the by-products of oxidation enabling the introduction of more methane and/or oxygen
- Selectively cooling the reference catalytic bead compared with the oxidising bead

Determination of the effect of ambient air velocities over-exiting (catalytic) sensors is again best undertaken in a controlled laboratory environment, where the effect of various air speeds and directions can be measured.

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1 The reference to this value in AS/NZS 60079-29-1:2008 has since been deleted from the AS/NZS 60079-29-1:2018 version.
2 This is equivalent to 5 mm of water at normal atmospheric pressure.
CONCLUSIONS

Given that ongoing detector performance is so reliant on correct calibration procedure, equipment and execution, it is argued that performance testing of detectors should include testing of the field calibration procedure.

Furthermore, the performance testing of calibration procedures should include all environmental conditions included in the performance testing of the detector itself. For example, if a given detector is performance tested in ambient air velocities up to 6 m/s, then the calibration procedure should also be tested under the same conditions.

If such performance testing was completed by an accredited laboratory as part of performance testing to a given standard, end users would have a higher degree of confidence in their application of the same procedure in the field.

The scenario painted begs the question as to how end-users can validate field calibrations. The following approaches are suggested:

- Ask your detector supplier to provide recommended field calibration kit and procedures.
- Ask your gas detector supplier if their field calibration kit and process has been performance tested under the same operating conditions as the detector itself.
- Validate the field calibration process in a laboratory against calibrated test equipment.
- Validate the field calibration process in the field by the use of a second (calibrated) detector to compare ‘before’ and ‘after’ results of the equipment under test.

REFERENCES


Standards Australia, 2018. AS/NZS 4641:2018, Electrical equipment for detection of oxygen and other gases and vapours at toxic levels - General requirements and test methods. SAI Global, Australia.


HOW BLOCKED GAS DETECTORS CHANGE
THE APPARENT CONCENTRATION OF GAS

Ian Webster

ABSTRACT: The operation of a diffusion type gas detectors used in fixed, machine mounted
and handheld applications is reliant on the natural equalisation of dissimilar gas concentrations
driven by partial pressures inside and outside the detector.

Typically, this equalisation is inhibited (to a greater or lesser degree) by protective filters and
barriers surrounding the fragile sensing elements from the typically harsh ambient
environments. The accumulation of dust and other foreign matter on the protective filters can
further inhibit the diffusion of gas into a detector.

The usual calibration process for a gas detector – typically by a ‘bump’ or ‘challenge’ test – will
often fail to detect when a detector is blocked, or partially blocked. This can lead to the
‘calibrated’ detector reading high or low, but with no way to determine if that is the case.

Retrospective examination of records and equipment from Pike River Mine lead to the
conclusion that critical detectors were affected by filter blockages, resulting in methane
detectors reading approximately one-half of the true concentration.

This presentation explores how a blocked detector can give an erroneous reading, and what
steps can be taken to avoid replicating previous mistakes.

INTRODUCTION

The Pike River Mine disaster in 2008 was investigated by multiple parties: the Royal
Commission of Inquiry (Royal Commission on the Pike River Coal Mine Tragedy, 2012) was
notably transparent in its deliberations and public in publishing its findings. A number of matters
relevant to gas detection safety systems were raised.

GAS DETECTORS AT THE PIKE RIVER MINE

The underground workings at Pike River were separated into ‘Restricted’ and ‘Non-Restricted’
zones, generally based on the likelihood of explosive gases (methane at 0.25% vol/vol) (New
Zealand Department of Labour, 2011). Equipment in the Restricted zone was required to be
explosion protected, while equipment in the Non-Restricted zone was generally not explosion
protected.

The perimeter of the Non-Restricted zone was monitored by a series of catalytic methane
sensors. See Figure 1. These detectors were configured to trip electrical power when the
ambient atmosphere reached 0.25% vol/vol methane. The efficacy of this means of protection
was arguably compromised by the close proximity of the detectors to the Restricted zone – in
some cases a distance of only several metres (New Zealand Department of Labour, 2011).
Such proximity made no allowance for the response time of the detector, nor for the inevitable
telemetry or tripping delays, given the flow of general body air forced by the mine ventilation.

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University of Wollongong, February 2019 250
Inside the Restricted zone, personnel and explosion protected equipment were guarded by three (identical) catalytic detectors configured to trip electricity supply at 1.25% methane.

Of particular interest were the two methane detectors, located respectively at the bottom and top of the main ventilation shaft. These two detectors monitored the same general body of air, albeit with a delay (associated with the transport lag of air moving up the ventilation shaft).

The detector at the bottom of the shaft was believed to have been mounted in a conventional manner, and readily accessible to maintenance workers. The detector at the top of the shaft was reportedly suspended down the shaft a distance of several metres, in an unidentified orientation. The top of the ventilation shaft was located in a generally inaccessible area, making checking and maintenance of the detector problematic. When last calibrated (prior to the first explosion), the sensor at that location was reported as ‘wet and muddy’.

Approximately eleven weeks prior to the first explosion, the detector at the base of the ventilation shaft was withdrawn from service. The control room console marked the detector as ‘Faulty – waiting for spare’. See Figure 2.

> There were several problems with the gas sensors in the ventilation shaft. First, the sensor at the bottom of the ventilation shaft stopped working on 4 September 2010, nearly 11 weeks before the explosion, and was never repaired or replaced. Indeed, the control room operator’s screen on the Safegas system was permanently annotated to say the sensor was ‘faulty’ and ‘waiting for spare’.

](Royal Commission on the Pike River Coal Mine Tragedy, 2012)
When compared to the last recorded data from the methane detector at the base of the shaft, the surface fan detector appears to be reading at half the value of the detector at the base, even though no dilution of the methane concentration was possible in the ventilation shaft. See Figure 3.

**Figure 3: Comparative methane detector readings at bottom (in blue) and top (in red) of main ventilation shaft at Pike River Mine**

(Royal Commission on the Pike River Coal Mine Tragedy, 2012)

In the absence of the identical detector previously located at the bottom of the shaft, this detector was the principal sensor measuring the concentration of methane in the general body of air.

**THE RECOVERED DETECTOR**

After the series of four explosion at the Pike River Mine, investigators somewhat fortuitously recovered the methane detector previously located at the top of the ventilation shaft. It had been ejected from the mine at some point, and was discovered in adjacent bushland.

The recovered detector had sustained superficial damage, but was otherwise intact. See Figure 4.

**Figure 4: Recovered methane detector from top of main ventilation shaft**
The display was found to be non-functioning. Connections terminals inside the enclosure were present, but damaged. See Figure 5. The sensing element of the methane detector was occluded by significant debris. See Figure 6.

Figure 5: Recovered methane detector from top of main ventilation shaft

Figure 6a: Sensing element occlusion on recovered methane detector

Figure 6b: Sensing element occlusion on recovered methane detector
The detector was subjected to a forensic performance examination. The detector was found to be operational, albeit reporting gas concentrations of approximately 50% of the applied concentration. This reading anomaly is consistent with the data from the mine site during operation. Significantly, the response increased after the debris was removed from the sensor housing.

The Gasguard Sensor (Serial No. 24063004) was inspected. Access to the enclosure was precluded by mechanical damage to one corner of the lid obscuring fixing screw. DII staff ground the lid to enable access to internal electronics. Blast damage had ripped external wiring through cable gland. DIN rail fittings were burned and fractured. Coal dust and charred remains were located inside enclosure. Some insulation in internal wiring was damaged and missing. Printed circuit boards appeared to be in relatively good condition. Identification labels were partially obscured. Wiring harness to catalytic bead was unplugged.

The sensor was powered from the controller, drawing ~25 mA load current with catalytic bead disconnected. This was considered normal. The liquid crystal display on the enclosure lid did not operate. The catalytic bead harness plug was re-inserted. Load current increased to ~72 mA – again normal. The sensor output was measured at 5.4 mA (expected to be 4 mA will zero methane).

The sensor output was connected to the controller input. With zero methane controller read 0.3%.

Methane gas at 1% concentration was applied. The sensor responded relatively slowly, rising to an indicated 0.43% (not steady state).

Methane gas at 3% concentration was applied. The sensor again responded relatively slowly, rising to an indicated 1.47% (not steady state).

The stone guard, hydrophobic barrier were removed, together with an accumulation of coal dust and debris.

Methane gas at 3% concentration was re-applied. The sensor again responded more quickly, rising to an indicated 2.06% (not steady state).

(Webster, 2014)

At the time of the examination, no further consideration was given to the anomalous readings, other than to note that the response time of the detector was inordinately slow.

RESPONSE TIME TESTING

The examination of the recovered detector prompted a hypothesis that occluded detectors would have slower response (t90) times than a ‘clean’ detector. The hypothesis was tested using an artificially occluded detector in a laboratory environment.

Figure 7 shows the measured response times. The conjecture that more severely occluded detectors have slower response times was confirmed.

Moreover, it was also observed that more severely occluded detectors also converged to lower indicated concentrations, even though the same concentration of test gas was applied in each instance.

This observation is consistent with the reported differences in detector readings from the bottom and top of the ventilation shaft at Pike River. The sensor at the top of the shaft was observed

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1 The t90 is defined as the time taken by a sensor to reach 90% of its final value.
to be compromised by debris, and the difficult access to that location could have limited frequent inspections and maintenance.

Figure 7: Detector response times with varying degrees of debris occlusion. Vertical axis is gas concentration (CH4% vol/vol); horizontal axis time time (secs)

WHY BLOCKED DETECTORS READ INCORRECTLY

Subsequent work has further examined the relationship between occlusion and measured accuracy.

Firstly, comparisons were made between catalytic type and Infrared (IR) types of sensing elements. It was observed that occluded IR detectors exhibit the same (slower) response times as do catalytic detectors, but do not deviate in (final) measured accuracy in the same way as do catalytic detectors.

Catalytic detectors, unlike IR technologies, consume methane during the sensing process. Such detectors require a constant flow of methane, and oxygen, to sustain the catalytic process. Furthermore, the by-products of the catalytic process need to be exhausted from the bead surroundings.

It follows, then, that there is a constant flow of gas into, and out of, a catalytic detector, unlike an IR where a static gas sample remain essentially unaffected by the measurement process. This gas flow in the catalytic detector is clearly vulnerable to impediment by accumulated debris on the protective housing and filtering system.

WHAT CAN BE DONE TO MINIMISE THE EFFECT OF OCCLUSION

The most obvious mitigation strategy to avoid compromise of detectors by occlusion is to keep the sensing elements (and associated filters) clean. This is of course problematic in underground coal mines where coal dust and stone dust are frequently encountered in general air bodies in both development and production areas.
Direct measurement of the effect of occlusion caused by accumulated debris is difficult in practice. A technique that measures differential pressure across filters and membranes caused by a known, constant flow rate of test gas has been demonstrated to quantify occlusion. See Figure 8.

Figure 8 Measurement of occlusion by pressure differential

The applicability of that test, however, is predicated on being able to access both sides (‘inside’ and ‘outside’) of the filter structure – a requirement that is generally not feasible in commercial detectors.

The demonstrated relationship between response time and reduced sensitivity of catalytic sensing devices also offers some insights.

Routine gas detector maintenance practices typically include a bump test whereby a test gas of known concentration is presented to the detector, and the detector adjusted to read that test concentration. Assuming that the calibration process itself does not impinge on the detector reading (say, by presenting gas at an increased pressure above ambient), this approach can arguably be used to compensate for the decreased sensitivity caused by occlusion. However, increasing the (amplitude) sensitivity of the detector does not address the compromise in detector response time. That is, an occluded detector may be adjusted to show correct concentration, but in the absence of direct measurement the response time can be compromised.

To this end, AS/NZS 2290.3 (2018) Maintenance of gas detectors in underground coal mines (Standards Australia, 2018) was recently revised to make response time testing a periodic requirement of the maintenance regime. The justification was two-fold:

- A compromised response time is itself a compromise of the fundamental safety function realised by the gas detector.
- A compromised response time in a catalytic detector is an indication of occlusion by debris.
Further work in the area is continuing through an ACARP\textsuperscript{1} project in association with Simtars and CMTS\textsuperscript{2}.

**OTHER ISSUES WITH GAS MEASUREMENT AT PIKE RIVER**

The various aspects of safety and operations at the Pike River Mine have been documented in the media and in the Royal Commission Final Report (Royal Commission on the Pike River Coal Mine Tragedy, 2012). Amongst these, one other aspect of the gas detection installation significantly compromised is reporting of true gas concentrations.

Figure 9 shows a portion of the methane concentration log from the surface detector. An extended duration of ‘flat-lining’ at a concentration of 2.96\% CH\textsubscript{4} is readily apparent. This behaviour was considered sufficiently anomalous to warrant further investigation.

![Figure 9 Portion of the methane concentration log from the surface detector](image)

The general configuration of the detector, controller, telemetry links and programmable logic controller (PLC) are shown in Figure 10. The detector and controller were both located in the Restricted Zone, and hence were explosion protected (intrinsically safe). The PLC was located outside of the Restricted Zone and, to maintain explosion protection, an intrinsically safe barrier was inserted between the controller and the PLC. The total resistance of the PLC and zener barrier was calculated and measured to approximately 550 Ohms.

Figure 11 shows the loop resistance versus loop current for controller. It is seen that the maximum loop resistance to enable full excursion of the loop current (to 20 mA) was approximately 330 Ohms. At the actual loop resistance of 550 Ohms, the maximum loop current was approximately 13.6 mA, which corresponded to a methane concentration of 2.96\%.

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\textsuperscript{1} Australian Coal Association Research Program, Eagle St, Brisbane City QLD.

\textsuperscript{2} Coal Mine Technical Services, North Wollongong, NSW.
Figure 10 General configuration of the detector, controller, telemetry links and PLC for surface methane detector
It follows, then, that the gas controller could never indicate a concentration of greater than 2.96%, notwithstanding any actual measured concentrations at the detector. Significantly, bump testing (calibration) of the detector was typically conducted at 2.0% – 2.5% methane, meaning that the telemetry deficiency was never revealed.

CONCLUSIONS

The series of explosions at the Pike River Coal Mine were the culmination of a large number of factors that impacted the operation and safety of the underground mine. Significantly amongst these, the accurate and timely measurement of instantaneous concentrations of explosion methane gas were found to be problematic.

One of the contributing factors was the accumulation of foreign debris into the environmental filters protecting the catalytic sensing element. The significance of this accumulated debris, and the effect on gas detection responsiveness and accuracy, were either not know, or not acknowledged.

A program of research has been initiated to quantify the effect this occlusion on gas detectors in coal mines, so as to inform the coal mining industry of the potential hazards and risks.

REFERENCES


AGEING EFFECT ON THE SELF-HEATING INCUBATION BEHAVIOUR OF LIGNITE

Basil Beamish¹, Jan Theiler², Andrew Garvie³

ABSTRACT: A lignite seam present in overburden strata of an opencut mine poses an interesting question as to its spontaneous combustion hazard likelihood when placed into an overburden spoil pile. Lignite is often assumed to rapidly spontaneously combust when exposed to air due to its low rank. However, lignite also has a high moisture content as-mined, generally in the order of 40% or greater. Self-heating is a balancing act between the intrinsic reactivity of the coal and the moisture present, which can act as a moderating influence to the rate of self-heating. In addition, this balance can be altered by ageing since both the intrinsic reactivity of the lignite and the moisture content decrease over time. This feature of coal self-heating is frequently overlooked in almost all spontaneous combustion test methods. A new adiabatic oven incubation test method is now routinely used by the Australian coal industry to overcome this deficiency. It assesses the spontaneous combustion hazard for the environmental conditions that exist for each mining situation. Incubation testing of the fresh lignite demonstrates that in an as-mined moisture state of 45.2% the incubation period of the lignite is considerably extended by heat loss from moisture liberation and evaporation to the point of no thermal runaway being achieved in a practical timeframe. However, rehandling or exposure to the atmosphere of aged overburden spoil containing the lignite within a certain timeframe can alter the heat balance in favour of thermal runaway. This paper presents laboratory examples of how this mechanism can occur.

INTRODUCTION

Spontaneous combustion poses a significant environmental and safety hazard to overburden spoil piles, particularly if remnant coal is present from rider seams in the overburden sequence. It is therefore necessary to evaluate the likelihood of developing a spontaneous combustion event from these stratigraphic units under a range of site conditions. This is often done by laboratory investigation of the ability of the coal to self-heat to the point of thermal runaway, which subsequently could lead to spontaneous ignition. There are many spontaneous combustion index tests used for rating the propensity of coal for spontaneous combustion (Nelson and Chen, 2007). Many of these have deficiencies and several ignore the fact that the development of a spontaneous combustion event is primarily an incubation process at low ambient temperatures (Beamish and Theiler, 2017). To overcome these deficiencies a new test method has been adopted by the Australian coal industry referred to as the Incubation Test, which is applicable to coals around the world ranging from lignite through to anthracite as well as non-coal strata (Theiler and Beamish, 2018).

Low rank coals are well-known to have a high spontaneous combustion propensity. However, they are still successfully mined and managed for this hazard. There is a common misconception that lignite in particular is highly prone to spontaneous combustion, but there are certain key factors that interact to moderate the likelihood of developing a lignite spontaneous combustion event including an ageing effect. This paper presents examples of these factors for a lignite seam that is present in the overburden sequence of an operating opencut mine. The
results obtained from adiabatic oven incubation testing clearly show the fine balance that exists between heat loss and heat gain in the low temperature region during self-heating incubation.

**LIGNITE SAMPLE AND ADIABATIC TESTING**

The coal quality details of the lignite sample are contained in Table 1. It has an ASTM rank classification of Lignite B and has a high ash content (>30%), but a low sulphur content (<1%). Both adiabatic $R_{70}$ self-heating rate and incubation testing as described by (Beamish and Beamish, 2011; Beamish, Edwards and Theiler, 2018) was conducted on the lignite in its fresh state as well as after one week of exposure to air and again after four weeks of exposure to air. The air exposure took place under controlled laboratory conditions using replicate splits of the lignite in a broken state.

Table 1: Coal quality data for lignite sample

<table>
<thead>
<tr>
<th>Sample</th>
<th>Moisture (% as-received)</th>
<th>Ash (% dry basis)</th>
<th>Sulphur (% dry basis)</th>
<th>Volatile Matter (% dry mineral matter free)</th>
<th>Calorific Value (MJ/kg, dry basis)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lignite</td>
<td>45.2</td>
<td>37.6</td>
<td>0.83</td>
<td>54.1</td>
<td>16.76</td>
</tr>
</tbody>
</table>

**ADIABATIC SELF-HEATING RESULTS AND DISCUSSION**

$R_{70}$ values of fresh and aged lignite

The $R_{70}$ values obtained for the fresh and aged lignite samples are shown in Figure 1 compared against lignite coal samples from New Zealand and Indonesia as well as sub-bituminous coal from Callide. The intrinsic spontaneous combustion propensity rating of the fresh lignite is extremely high based on the classification scheme published by Beamish and Beamish (2011). The high ash content of the sample creates both a dilution and heat sink effect on the self-heating rate, which can be seen by comparing against the $R_{70}$ value obtained for the low ash content lignite samples from New Zealand and Indonesia. This ash content trend for the lignite samples has a similar slope to the sub-bituminous Callide coal trend seen in Figure 1 and published by Beamish and Blazak (2005).

![Figure 1: Relationship between $R_{70}$ self-heating and ash content showing intrinsic spontaneous combustion propensity ratings](image)
As the coal ages, the R70 value initially decreases quite dramatically due to the loss of reactive sites from oxidation. This is shown in Figure 2 and is due to the open macropore structure associated with lignites that enables ease of oxygen access to reactive sites. The R70 value decreases with exposure time, and would most likely approach a residual value. This is due to the greater degree of difficulty for oxygen to penetrate the finer micropore structure of the lignite after the majority of the macropore reactive sites have been deactivated. Beamish, Barakat and St George (2000) measured a similar non-linear decreasing trend in R70 value with ageing for a New Zealand sub-bituminous coal.

![Figure 2: R70 self-heating rate decrease with exposure time to air](image)

**Incubation behaviour of fresh and aged lignite**

Incubation testing of the fresh and aged lignite enables a greater understanding between the interaction of the intrinsic reactivity of the lignite and the moisture that is present for each situation. The fresh lignite gradually self-heats (Figure 3), consistent with the extremely high intrinsic reactivity as the oxygen gains access to the readily available reactive sites. During this process there is also heat loss as a result of moisture liberation from the coal and subsequent evaporation. After reaching a maximum temperature of approximately 90 °C, the heat loss by evaporation exceeds the heat gain from oxidation and the temperature decreases with no thermal runaway being recorded. This incubation feature of fresh lignite is not well-documented by previous research due to the limitations imposed by previous test methods.

After one week of ageing in air, the moisture content of the lignite reduces from 45.2% to 26.7%. The R70 value, which is a measure of intrinsic reactivity, decreases from 25.33 °C/h to 8.11 °C/h. This change in the lignite properties has a significant effect on the incubation self-heating behaviour. The low temperature self-heating rate is much faster than the fresh lignite (Figure 3), because the heat loss from the moisture liberation is substantially reduced. The sample temperature increases to approximately 96 °C with a subsequent balance of heat generated through oxidation and heat loss through evaporation. This produces a prolonged moisture evaporation temperature shoulder. Eventually, sufficient moisture is removed from the lignite to enable oxygen access to fresh reactive sites and self-heating continues at an increasing rate to thermal runaway.
Figure 3: Adiabatic oven incubation self-heating curves for fresh and aged lignite

After four weeks of ageing in air, the moisture content of the lignite reduces to 11.3% and the corresponding $R_{70}$ value is 4.12 °C/h. In this more aged state the incubation self-heating is much slower at low temperatures due to the loss of intrinsic reactivity from oxidation. However, as the coal temperature increases the heat loss from moisture evaporation is diminished compared with that of the fresh sample due to the lower starting moisture content and only a small decrease in self-heating rate is observed between 100-110 °C, before self-heating resumes to accelerate to thermal runaway. Consequently, in terms of overall incubation period the four week aged lignite has a shorter incubation period than the one week aged lignite.

Clearly as the lignite ages further, the intrinsic reactivity continues to decrease and a point is eventually reached where the incubation period begins to increase and no thermal runaway is possible. This is confirmed in recent work on aged coal characterisation at Leigh Creek Mine in South Australia (Garvie et al, in press).

CONCLUSIONS

Spontaneous combustion of freshly mined lignite is a complex process that is controlled by the interaction between heat release associated with the extremely high intrinsic reactivity of lignite and the heat loss due to liberation and evaporation of its initial moisture content. The process is one of self-heating incubation from low ambient temperature until either thermal runaway is achieved or the heat loss from the moisture liberation and evaporation dominates causing a decrease in temperature over time. If the lignite is left exposed to air and rehandled or if it is placed in a spoil pile that is later exposed to an air source, then self-heating to thermal runaway is possible due to a change in the heat balance controlled by the intrinsic reactivity of the lignite and its moisture content at the time of exposure. There is a range of material ages and moisture contents for which thermal runaway may occur that can be evaluated using incubation testing of the lignite.

REFERENCES


MANAGEMENT OF SPONTANEOUS COMBUSTION IN COAL OVERBURDEN SPOIL PILES

Andrew Garvie¹, Ken Donaldson², Basil Beamish³, Brad Williams⁴, John Chapman⁵

ABSTRACT: Laboratory testing was undertaken to examine the spontaneous combustion propensity of some of the coal seams being mined at Leigh Creek. Tests indicated that while carbonaceous rocks retained a fuel load, in isolation, these did not have the capacity to reach thermal runaway. However, the potential existed for heat from another source, such as coal, to raise the temperature of these rocks to above the threshold for thermal runaway (>100 °C).

The mine closure plan submitted to the regulator (Department of Premier and Cabinet, South Australia) incorporated a monitoring trial of the selected spontaneous combustion management rehabilitation strategy to demonstrate its effectiveness. The strategy included reducing batter slopes of waste spoil piles and the application of an inert cover. The trial was established in June 2017 in a location with active combustion immediately prior to rehabilitation treatment. Measurements of temperature and oxygen concentrations within the spoil pile over twelve months show that oxygen is consumed within 1 m of the outer surface of the waste, while maximum spoil pile temperatures have been decreasing, indicating a net heat loss from the trial spoil pile area. No spontaneous combustion outbreaks have occurred in the trial area since the trial commenced. Characteristics of the trial area materials, the management strategy, and outcomes from the field trial measurements are presented.

INTRODUCTION

The Leigh Creek Coal Mine, located approximately 500 km north of Adelaide in South Australia, operated between 1944 and 2015. During operations, localised spontaneous combustion occurred on numerous occasions in the overburden spoil piles containing relatively small volumes of coal (about 0.8%). The majority of the overburden sequence consisted of sandstones and carbonaceous mudstones. Samples of overburden rocks had total sulfur and total organic carbon present at up to 2.2 wt% and 14 wt% respectively.

The coal at Leigh Creek mine is a low rank, sub-bituminous brown coal that is prone to self-heating and spontaneous combustion. Spontaneous combustion also occurred in the spoil piles throughout the decades of the mining operation. Consequently, spontaneous combustion management strategies suitable for short-term control were developed, but were not specifically tailored for long-term closure.

The Alinta Energy Board announced the closure of the Leigh Creek Coal Mine in June 2015. The announcement was preceded by a brief period of internal planning for closure. Coal mining operations ceased in November 2015.

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University of Wollongong, February 2019
A swift decision to close, followed by a short mining operations shutdown period, presented significant technical and regulatory challenges for both the mine operator and regulators. Subsequent to the decision to close, a joint risk mapping process was undertaken by the operator, Flinders Power, and the South Australian mining regulators to obtain objective evidence to inform the risk profile and risk management strategies for closure. Spontaneous combustion occurrences in the waste spoil piles were identified as a significant risk.

Three factors potentially contributing to and/or suitable for the management of spontaneous combustion were identified as requiring further investigation during the risk assessment:

- The coal and the carbonaceous mudstone overburden both have the ability to self-heat to the point of thermal runaway.
- The ability of coal and mudstone to self-heat to the point of thermal runaway reduces with time of exposure.
- Reducing the batter angle of the spoil pile and covering with a layer of fine inert material would reduce the risk of spontaneous combustion.

**MATERIAL CHARACTERISATION**

Characterisation of the overburden materials at Leigh Creek was conducted in two separate stages.

The initial stage involved the collection of grab samples from various locations across the mined-out area covering a range of overburden rock types and ages, including aged coal. Geochemical analysis of the total organic carbon and total sulfur contents of the non-coal overburden samples was conducted to provide a preliminary assessment of their potential to spontaneously combust and to help select appropriate samples for more specific testing. The sulfur content was found to range up to 2.2% total sulfur and the organic carbon ranged up to 14%. The high total sulfur contents in this series of samples were identified from XRD analysis to be associated with gypsum.

The R$_{70}$ self-heating rate (Beamish, Barakat and St George, 2000) was determined for selected coal and non-coal overburden samples. The results are shown in Figure 1. All of the non-coal overburden had R$_{70}$ values less than 0.35° C/h, placing them in the ‘Low’ intrinsic spontaneous combustion propensity rating (Beamish and Beamish, 2011). These samples had ash contents in excess of 75% (Figure 1) and spoil with these ash contents would require an external heat source, or the presence of reactive pyrite, to elevate the temperature of the material to the point where thermal runaway would be possible.

The aged coal had R$_{70}$ values much lower than the equivalent fresh coal samples (Figure 1) due to *in situ* oxidation. The values correspond to a ‘Low-Medium’ to ‘Medium’ intrinsic spontaneous combustion propensity rating. In this oxidised state, the coal does not have sufficient intrinsic reactivity to overcome the heat loss associated with evaporation of moisture. This heat balance effect (Beamish and Theiler, 2015) is demonstrated for fresh Leigh Creek coal samples tested at two different moisture contents (Figure 2).

From these initial results it was determined that, historically, the most likely source of spontaneous combustion in the overburden spoil piles at Leigh Creek was fugitive coal mixing with the carbonaceous non-coal overburden and forming hot spots sufficient to initiate sustained self-heating in the overburden pile.
Figure 1: Self-heating relationship with ash content for fresh coal sample (BH7030) compared against aged coal (SCP020, SCP051 and SCP036) and non-coal overburden strata from Leigh Creek Mine. Annotation above aged coal samples is in years.

Figure 2: Coal self-heating curves for Leigh Creek coal sample at two different moisture contents showing the balance between intrinsic reactivity and moderating effect of moisture.

The second stage of characterisation was conducted on drilled samples from the rehabilitation trial area on the spoil pile. The results are shown in Figure 3.

One of the samples returned a total sulfur content of approximately 4% (the two values in Figure 3 are repeat measurements on the same sample). The pyritic sulfur content of the sample was
approximately 3.5%, as determined from XRD analysis, present predominantly as marcasite (6.4%) and, to a lesser extent, pyrite (3.8%) contained in stringers cutting through siderite nodules (Figure 4). The total organic carbon content of the sample was approximately 2% and consequently, the R70 value was effectively 0.

![Figure 3: Relationship between total sulfur content and total organic carbon content of rehabilitated overburden samples from Leigh Creek Mine](image)

![Figure 4: Photograph of siderite nodules containing marcasite/pyrite stringers](image)

Incubation self-heating test results are shown in Figure 5. The majority of the overburden samples lost heat due to evaporation of residual moisture and therefore thermal runaway is not possible in these types of spoil unless they are in contact with other material at temperatures in excess of 100 °C. Sample H6-28 was at an elevated temperature of 110 °C in the spoil pile; consequently, if it was exposed to an air source it could easily progress to thermal runaway. The sample containing marcasite/pyrite (H3-4) was tested at two moisture contents (3.6 wt % and 11.0 wt %). At the elevated moisture content, the sample rapidly self-heats from the marcasite/pyrite oxidation reaction, but then reaches the point where heat loss from moisture evaporation takes over and the sample eventually cools down.

At the lower moisture content, however, the sample rapidly self-heats then slows as it begins to liberate and evaporate moisture, but then reaches the point where the organic carbon begins to oxidise and sufficient heat is generated to progress to thermal runaway. This situation is very
localised in the overburden spoil pile (detected in one of approximately 200 samples) and can only occur if the pyrite is liberated from the siderite nodules. This could only take place if, for example, heavy machinery crushed the nodules during emplacement or re-profiling of the overburden spoil pile, since the siderite nodules are extremely hard.

![Figure 5: Incubation test results for rehabilitated overburden samples from Leigh Creek Mine](image)

**GAS TRANSPORT**

Three mechanisms, diffusion, convection and advection, can transport significant quantities of oxygen from the surface to the interior of an overburden spoil pile. Diffusion is driven by a gradient in the partial pressure (or concentration) of oxygen within the spoil pile. Advection and convection are driven by spatial gradients in the total gas pressure and buoyancy, respectively. Causes of advection include wind or atmospheric pressure variations. Buoyancy arises due to temperature gradients within the spoil pile and is most readily established in the batter of a spoil pile where warmer rising pore gases are more readily replaced with air lower in the spoil pile. Therefore, reducing the batter slopes tends to reduce the likelihood of convection.

The material property that controls advection and convection is the intrinsic permeability; the property that controls oxygen diffusion is the effective oxygen diffusion coefficient. Compaction of spoil may provide a significant reduction in the intrinsic permeability but is less effective in reducing the effective oxygen diffusion coefficient. The effective oxygen diffusion coefficient decreases significantly as the degree of moisture saturation of the material increases above 0.85.

**Management strategy**

The management strategy selected for the spoil piles at risk of spontaneous combustion was to reduce the batter slopes by incrementally pushing waste from the crest to towards the toe using D10 bulldozers. Multiple dozer movements also affected a reduction in the permeability of the near-surface waste. The slopes were then covered with a compacted layer of inert soil materials sourced locally. The batter slopes were reduced from approximately 1:1.3 (V:H) to...
within the range of 1:4.5 to 1:5.0 (V:H). The single layer soil cover, after compaction, had a minimum thickness of 1.2 m.

**Field trial**

A field trial to quantify the effectiveness of the management strategy was established on the batter of a spoil pile representative of spoil piles at risk of spontaneous combustion, as indicated by elevated surface temperature. The trial area was resloped on 24 May 2017 and covering was undertaken between 14 and 19 June 2017. After resloping and before covering, the highest measured surface temperature was 121 °C. No cover was placed on the horizontal bench above the trial batter.

Instruments were installed along five vertical drill holes (Figure 6) to measure pore gas oxygen and carbon dioxide concentrations and spoil pile temperatures. Four of the locations were on the batter and the fifth was in the flat top of the spoil pile. The monitoring equipment configuration for each drill hole is illustrated in Figure 7. Small diameter gas sampling tubes were installed to extend from the spoil pile surface and terminate at different depths. A string of temperature sensors was hung permanently inside the pipe (approximately 45 mm diameter). Oxygen and carbon dioxide concentrations were determined by pumping small volumes of pore gas from the spoil pile via the gas sampling tubes to portable gas analysers. Measurements were made multiple times in the week after instrument installation and then approximately monthly thereafter.

![Figure 6: Monitoring locations in plan view designated H2 to H9](image)

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Oxygen introduced to the waste during sonic drilling of the instrumentation drill holes penetrated to about 15 m (Figure 8a, 24/6/2017) and most was consumed within a week. Subsequent oxygen concentration profiles show that oxygen ingress was limited to approximately 0.5 m below the cover, consistent with oxygen supply into the dump by diffusion and oxidation occurring in the 0.5 m of layer of waste below the cover contact. One week after installation, the oxygen concentrations were steady (Figure 8b).

The highest temperature of 138 °C was measured about 13 m below the surface at location H6 (Figure 8c). The absence of oxygen at this location indicates that oxidation was occurring at this depth before the monitoring began. Furthermore, the decrease in temperatures at all depths at location H6 over time (Figure 8d) indicates heat loss from this region and that the overall oxidation and heat generation rates have decreased or ceased. The time for the maximum temperature to decrease below 100 °C, the temperature required for thermal runaway, was estimated from measured temperatures to be about 460 days after the first measurements.

Carbon dioxide concentrations were high throughout (Figure 8e). The sharp change in the CO₂ concentration gradient at about 1.5 m indicates that CO₂ was produced at about this depth. This is consistent with the depth at which oxygen was being consumed. The approximately constant CO₂ values below 1.5 m indicate that the likely rate of CO₂ production is low or zero below this depth.

Inferred oxygen concentration contours for the cross section of the spoil pile through H3, H6 and H9 (Figure 8f) are consistent with oxygen supply by diffusion to the outer 2 m layer of the spoil pile. Temperature contours (Figure 9) indicate the presence of a hotter volume at about 14 m below the surface. That volume has cooled due to heat loss to adjacent spoil material and loss from the upper and lower boundaries (surfaces).
Figure 8: Oxygen carbon dioxide concentrations and temperatures measured at location H6

Figure 9: Inferred temperature contours in a trial area cross section
SUMMARY AND CONCLUSIONS

Samples of the small quantities of aged coal in the spoil piles had ‘Low-Medium’ to ‘Medium’ intrinsic spontaneous combustion rating (much lower than the equivalent fresh coal samples), and did not have sufficient reactivity to overcome the heat loss associated with moisture evaporation. Therefore, it is concluded that the coal in the spoil piles would be not likely to reach thermal runaway via self-heating.

Most non-coal spoil is carbonaceous mudstone. The carbonaceous mudstone presents a fuel load, but would require an external heat source, or heat from the oxidation of reactive pyrite or marcasite, that may raise temperatures to in excess of 100 °C, for thermal runaway to occur. Occurrences of the sulfide minerals pyrite and marcasite are infrequent (1 in 200 samples) and typically are bound within high strength siderite nodules. Should the siderite nodules be crushed and the sulfide minerals become exposed to oxygen, it is possible that, at certain moisture contents, sulfide oxidation could raise the temperature enough for the organic carbon to self-heat and possibly reach thermal runaway.

Oxygen concentration and temperature distributions were not measured before the batter of the trial region was resloped and covered. However, elevated temperatures within the trial area (up to 138 °C) are evidence of combustion prior to establishment of the trial area. Since the resloping and covering, oxygen penetrated less than 1 m into the waste below the cover. The observations indicate that the dominant oxygen supply mechanism is diffusion, with no evidence of convection being observed within the monitored region.

A consequence of limited oxygen ingress is the cooling of the hot volume within the trial region. Based on cooling rates estimated from the observed temperature changes over time, the maximum temperature in the monitored region would be expected to decrease to below 100 °C (the trigger temperature for thermal runaway) about 460 days after the first measurements.

The monitoring results indicate that the resloping and covering have been effective in limiting oxygen supply into the spoil and have allowed temperatures in most of the underlying material to decrease, thereby reducing the likelihood of ongoing spontaneous combustion.

REFERENCES


PERFORMANCE ANALYSIS OF SURFACE GOAF GAS DRAINAGE HOLES FOR GAS MANAGEMENT IN AN AUSTRALIAN COAL MINE

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ABSTRACT: This paper focuses on assessing gas drainage performance from vertical surface goaf holes in an operating mine with high specific gas emission rates (~20 m³/t). Field monitoring results of gas flow rate and gas composition collected from individual gas drainage holes were analysed. The major challenge of implementing the full potential of goaf drainage system is the perceived risk of spontaneous combustion (sponcom) in goaf using arbitrary trigger set points. Although the provenance on the introduction and reasons for the use of CO is unknown, the paper highlights that the current use of CO based trigger action response plan (TARP) for sponcom management leads to the premature closure of goaf holes, which largely constrains goaf hole gas drainage performance and has a significant bearing on longwall gas management. The CO concentration in gas captured by goaf holes is found to be positively correlated with O₂ concentration and negatively correlated with CH₄. Building upon abundant field measurement data, goaf gas profiles for CO, O₂, CH₄ and CO₂ concentration were established, which suggest that the increase trend of CO level behind the face is a normal behaviour of goaf closure and would recover to trivial concentration after 300 m deep into the goaf. The paper provides the basis to eliminate or review the use of CO triggers in current surface goaf gas management TARP levels of longwall panels, which has detrimental effect on longwall TG gas management for explosion prevention.

INTRODUCTION

Coal has been a primary source of global energy production and development for the past two centuries and is expected to continue into the near future. Today, coal supplies 40% of global electricity, and in some countries such as South Africa and China, it supplies over 90 % and 70% respectively of the electricity generating source. In the case of Australia, its association with coal can be traced back to as early as 1797. Methane is a major gaseous constituent of the coal seam. It gets released during mining and can result in unsafe working conditions.

Methane is explosive in the range of 5% to 15% in air, its transport, collection, or use within this range, or indeed within a factor of safety of at least 2.5 times the lower explosive limit (2.0%) and at least two times the upper limit (30%), is generally considered unacceptable because of the inherent explosion risks. Methane is contained under pressure within the micropores and fractures of coalbeds and in the adjacent strata and gets released into the mine atmosphere during mining (Moore, Deul and Kissell, 1976). The removal of methane by conventional ventilation is particularly difficult in Longwall (LW) mining of gassy coalbeds, which are prone to dangerous outbursts. In Australia, typically, pre-drainage of working seam would entail management of the outburst risks by ensuring the gas content values below 7 m³/t as one of the rule-of-thumb outburst risk indicators. A combination of pre- and post-drainage using advanced, Surface-based In-Seam (SIS), Medium Radius Drilling (MRD) and Underground In-Seam (UIS) directional drilling techniques are utilised in Australia (Belle, 2017).

Goaf is a broken and loose, highly permeable ground where coal has been extracted by coal mining and the roof has been allowed to collapse, thus fracturing and de-stressing strata above
and, to a lesser extent, below the seam being worked. In a typical U ventilated longwall coal mine, the goaf gas is generated from the broken and left over coal as well as the liberation from the upper and lower coal seams adjacent to the working seam. When these goaf gases are not managed, they will enter into the general body of the ventilation return air by means of goaf fringe on the tailgate side of the longwall. It is important to note that with limited longwall ventilation dilution capacity, ensuring adequate and timely goaf drainage is critical. In multi-seam longwall mining, goaf drainage plays a fundamental and critical role in managing the gas levels. The volume of gas in the post mining depends on the thickness of upper coal seams, gas content, rate of longwall retreat, gas purity, magnitude gas domain. Generally, the volume of goaf gas is significantly higher than the pre-drainage of the working seams. Typically, goaf drainage holes used are as follows, viz., horizontal boreholes, cross-measure boreholes and vertical surface goaf holes. For longwall retreat mining, surface vertical goaf boreholes are drilled from the surface into the upper limits of the goaf, collecting goaf gas from the de-stressed goaf area. Vertical goaf holes are the most appropriate, efficient and safe methodology where there is surface access to an operating longwall mine. Where there was a failure of surface goaf holes, attempts have been made to access the goaf space using 203 mm (8 in) drainage pipes through perimeter roadway seals (Belle, 2017) under controlled Trigger Action Response Plan (TARP) values for methane purity and oxygen levels for safe and effective longwall mining.

The provenance of the introduction of CO (carbon monoxide) as a trigger for sponcom management is unknown or not well documented. Anecdotal discussions suggest that they are an indication of sponcom activity in the active and sealed longwall goaf. It is measured and documented the presence of elevated levels of oxygen behind the goaf as a result of the used ventilation system. The presence of this oxygen under very humid and hot conditions enable the left-over broken coal behind the longwall chocks leading early coal oxidation. If not monitored and controlled, this condition may turn itself into elevated levels of heating leading to sponcom fire. Therefore, to prevent potential sponcom hazards, underground seal gas compositions are measured for sponcom indicator gases.

The current goaf gas drainage practice in Queensland mines also monitor the gas composition in particular CO levels in individual goaf holes to stop the goaf hole operation. Although CO level in ventilation air acts as a robust trigger value for sponcom control in underground ventilation system, it is unclear whether CO level in drained goaf gas also would provide an accurate trigger for sponcom incidents in the goaf. Building upon the analysis of gas drainage performance from goaf holes at a high production gassy mine, this paper reviews the performance of goaf drainage holes and their gas composition to manage the tailgate gas levels as well as reviews the relevance of the current CO-based TARP for sponcom management in goaf holes.

GOAF HOLE GAS DRAINAGE AND SPONCOM TARPS

This section of the paper provides the background to the goaf hole spacing and their performance in managing the methane gas levels in a high production longwall operating mine. In the recent panels, in order to maintain ventilation and gas regulatory compliance, a disciplined data collection system was established, which includes total goaf gas flow and composition from each of the goaf holes, underground longwall real-time gas, ventilation airflow, barometric and production. These have enabled operators to understand the goaf gas dynamics, location of gas zones and anticipate controls and optimise future drainage programs. The results and analyses carried out on three longwall panels (LW6, LW7, and LW8) are based on this data collection system, with particular reference to goaf gas flow, gas composition and use of CO levels as a trigger for sponcom management. Typically, the manual goaf hole gas composition parameters are used to slow down or shut-off individual goaf hole operation.
This paper will focus on analysing data collected from a representative panel LW7 along with additional two LW panels. There are in total 92 goaf holes in the LW7 panel. Among these goaf holes, 71 were drilled a 30-40 m away from the tailgate roadway with ~50 m spacing, while 21 holes were drilled 50 m away from the maingate (MG) with ~150 m spacing (Balusu, 2016). From inbye to outbye, the goaf holes at the Tailgate (TG) side were named as 7-01 to 7-71, and the goaf holes at the MG side were named as 7-MG01 to 7-MG21. Typical goaf holes would have been fully cased in the top portion and the bottom 48 m of the holes were cased with slotted pipes to allow gas flow into the pipes. The diameter of these goaf holes was 250 mm (10") and their length was 330 m with bottom hole completion depth at 10 m above the coal seam. The first tailgate goaf hole was approximately 25 m from the start-up line.

GOAF HOLE PERFORMANCE ANALYSIS

Overall performance

![Figure 1: Cumulative flow volume in individual goaf holes from inbye to outbye at LW7](image)

The following paragraphs provide analyses of LW7 goaf hole performance analyses. A large variation of cumulative flow volume in individual goaf holes can be observed across the entire LW7 panel (Figure 1). Goaf hole 7-20 and 7-MG19 achieved more than $5 \times 10^6$ m$^3$ methane production while in goaf holes such as 7-04 and 7-58, the cumulative methane flow volume was less than $5 \times 10^4$. Average methane flow rate and methane concentration of drained gas in individual goaf holes are shown in Figure 2, which also suggests a large variation of the purity and efficiency of goaf gas drainage in different holes. A general increase trend of methane purity can be observed as goaf holes move from inbye to outbye at both TG and MG sides of this panel. Gas drained from tailgate goaf holes ranging from 7-01 to 7-19 at the start-up stage shows low flow rate and methane purity due to incomplete goaf cavity formation and lower Specific Gas Emission (SGE) levels along the inbye portion of the panel. On the other hand, methane flow rate in goaf holes 7-47 to 7-63 is relatively low, but they have much higher methane purity. The lower flow rate could be attributed to hole blockage and failures.
Goaf holes were opened sequentially along with the progress of longwall face retreat. There is no standard prescribed procedure to shut-off these holes, but CO is seen to be the main reason for premature shut-off. The average active production time for each goaf hole was approximately 21 days as shown in Figure 3. Note that a certain number of goaf holes with extremely short active days, particularly 7-01 to 7-19, were result from the high concentration of CO observed (>60 ppm) in those goaf holes. This led to the premature closure of those holes due to the concern of sponcom activity and the lower CO trigger values used in the goaf hole TARPs. Although flow rates were much lower as observed in goaf holes from 7-47 to 7-63, a much longer active/production days (>40 days) was achieved in these holes due to high methane purity and low CO presence. Note that two goaf holes (7-20 and 7-MG19), which achieved the best gas drainage performance, accredited to the long production period, stable methane flow rate (>400 l/s) and high methane purity (~90%).
The number of active goaf holes in each coal production day during the lifespan of LW7 panel is shown in Figure 4. As explained in the previous figure, in the start-up stage between 17 July to 17 Sep 2017, the low life span of goaf holes results in less active holes (~6) and higher CH₄ concentration observed in the tailgate (~1.25%). On the other hand, there were 14 active goaf holes in average between 2 Dec 17 to 2 Feb 18, which significantly reduced CH₄ concentration in tailgate to ~0.75%. This further demonstrates that with increased size of the goaf gas reservoir, it is important to increase the number of deployable goaf holes to manage the TG gas levels.

Histograms of average methane flow rate and average methane purity in each goaf hole are shown in Figure 5. The average methane flow rate in over three-quarters of goaf holes is higher than 400 L/s and nearly 65% of goaf holes presented methane purity higher than 70%. In general, compared with the TG side, goaf holes drilled at the MG side have better gas drainage performance in terms of higher methane flow rate and methane purity. This can be attributed due to MG holes having access to larger source of gas reservoir from adjacent undrained panels.

Methane production curves for a number of goaf holes are shown in Figure 6, where side by side (linear neighbouring) goaf holes were plotted. In general, neighbouring goaf holes
presented similar gas production trend. Among all goaf holes, a sharp increase of methane production rate was observed within the first five production days. Peak flow rate normally achieved at the fifth production day and then was followed by a gradual decline until the 20th day. If the goaf hole was still active, a long and stable tail with around 400 l/s methane flow rate would be generally observed. The long and stable tails from these methane production curves suggest most of the goaf holes in LW7 were prematurely switched off using either CO TARP or the need to deploy the well-head for the next outbye hole, were seriously under-performing and there is significant potential to increase goaf hole performance by extending their drainage lead time and thus assisting the management of longwall TG gas levels.

Figure 6: Methane production curves in side by side neighbouring goaf holes

GOAF HOLE GAS COMPOSITION ANALYSIS

The following paragraphs provide an analysis of gas composition of monitored goaf holes. For each goaf hole, the daily captured gas composition was manually monitored over its production period. Heatmaps illustrating the concentration of CO, O₂, and CH₄ for all the tailgate and MG side goaf holes are shown in Figure 7. Each small colour-coded block in these heatmaps represents gas level for a specific goaf hole at one production day. Non-active goaf holes or production days are not coloured. Thus, for each goaf hole, the horizontal length of coloured blocks represents the active operation time (days) of that hole. Similarly, at each production date, the vertical length of coloured blocks represents the number of active holes on that specific day.

As expected, a strong correlation between CO, O₂, and CH₄ levels can be observed, i.e., O₂ percentage is positively correlated with CO and negatively correlated with CH₄. As expected, at the start-up stage of the panel, a high percentage of O₂ was repeatedly monitored in corresponding surface goaf holes, in particular at the TG side, which coincided with the higher levels of CO and consequently the early termination of these holes as a result of TARPs. Higher levels of CO and O₂ measured during the goaf hole start-up is merely a reflection of goaf gas composition immediately behind the longwall face of coal oxidation. It may be possible that the higher velocity of goaf gas flowing past the upper seams may also be contributing to CO generation. There is a misperception that the CO levels recorded is an indicator of elevated levels of sponcom activity in the active goaf. However, based on the recent sponcom incidents of coal mines in Australia, it is suggested that the measured CO levels are an order of up to 1000 ppm higher than the levels recorded from the active goaf drainage holes. Similarly, the CO levels recorded in the Goonyella longwall seam are higher than the German Creek seam.
longwall. Note that the goaf hole 7-20 was shut-off at the early stage due to high CO but reopened after two months and only a negligible amount of CO was found after re-opening the goaf hole. From mid of October 2017, O₂ level was largely reduced along with a notable improvement in CH₄ purity and production time. During the panel completion stage (early February 2017), another CO increase trend was observed in outbye goaf holes 7-64 to 7-71. Note that high CO or low CH₄ blocks tend to cluster at neighbouring goaf holes or consecutive production dates, which indicates the spatial and temporal continuity of drained gas quality.

The average CO, O₂ and CH₄ concentration over the lifespan of individual holes are shown in contour plots in Figure 8, where the correlation between three gases can also be observed. Goaf gas drainage data collected from LW6 and LW8 were also used here to generate these contours. Note that tailgate goaf holes close to the face start-line (within 800 m outbye) tend to have high CO, high O₂ and low CH₄, which reflects the challenging CO condition during the start-up period and they are certainly not reflective of heightened spontaneous combustion activity. Also, there is no correlation between underground bag samples from sealed areas and goaf hole CO levels. Similar gas composition was observed in goaf holes next to each other, which suggests the gas drainage performance for goaf holes are also spatially correlated.
A closer view to examine the goaf holes which experienced high CO issues is shown in Figure 9, where the CO level increased gradually over time and a similar increase trend of O\textsubscript{2} ingress and decline trend of CH\textsubscript{4} purity were also observed. With the presentence of 10% O\textsubscript{2} in the start-up stage goaf holes, it took approximately 15 days from the first observation of CO until the CO concentration reached the goaf hole shut-off threshold. While for completion stage goaf holes with lower O\textsubscript{2} level, a much longer oxidation time was required, and the maximum CO level was also lower. Compared with the delayed response of CO level, the presence of O\textsubscript{2} or the drop of CH\textsubscript{4} purity suggest that the longwall retreat gas reservoir area is of low rate of gas emission and goaf hole is operating just behind the longwall faceline collecting the oxygen rich goaf gas. It is to be noted that in these longwall panels, there is no large volume of coal gets left in the longwall goaf. Based on the goaf gas analyses of historic 17 LW panels, the use of CO levels from goaf holes have not provided any relationship between underground sponcom activities. On the other hand, the use of lower level CO triggers based on coal gas evolution tests have resulted in premature shut-off the goaf holes resulting in elevated TG gas levels.
Figure 9: Gas level trends in goaf holes with high CO concentration issues.

Goaf gas profiles

The gas composition measurement results from goaf holes at different distances pertinent to the longwall face during the start-up period (July to September 2017) and completion period (Dec 2017 to Feb 2018) are plotted in Figure 10 and Figure 11, respectively. A clear trend of gas profiles can be observed in the goaf during these periods, which are consistent with the field observations. As discussed earlier, the observation of high CO level led to the premature shut-off of goaf holes, which largely limited their degasification effect. However, according to goaf gas profiles, the presence of high CO level is most likely due to the natural behaviour of goaf closure, which is characterised by an active zone close to the face and an inert zone far deeper in the goaf.

From the gas profiles in Figure 10 and Figure 11, it can be concluded that the active zone is around 300 m wide and the inert zone is from 300 m beyond in the deep goaf and these would vary from mine to mine. As moving deeper into the goaf, the active zone shows an increase trend of CO and CO₂, a relatively high level of O₂, and a mix of CH₄ level response. The gradual increase of CO level in the active zone is a reflection of normal coal oxidation in an oxygen-rich and high moisture and hot goaf environment. On the other hand, in the inert zone, a general declining trend of CO and O₂ can be observed, together with an increase of CH₄ and CO₂ concentration in deeper goaf. This indicates the continued emission of methane from the upper and lower destressed coal seams in the deep goaf and depletion of O₂, which leads to the progressive decline of the CO level.

The identification of the active zone and its behaviour is of significant importance for the goaf gas drainage plan since it defines the ‘normal’ trend of gas behaviour in the goaf. The observed relatively high CO level in goaf holes may not be a sufficient shut-off trigger if these holes are in the active zone (ranging from 100 to 300 m behind the face). Furthermore, it is imperative to extend the degasification period of goaf holes since once these holes are 300 m behind the face (in the inert zone), they can produce a much purer methane flow with no association to sponcom activity.
Figure 10: Goaf gas profiles from goaf holes at different distances pertinent to the longwall face during the start-up period (17/07/2017 to 17/09/2017).

Figure 11: Goaf gas profiles from goaf holes at different distances pertinent to the longwall face during the completion period (12/12/2017 to 12/02/2018).
DISCUSSION

The current operational time for most goaf holes is too short with an average gas production period of 21 days with some holes operated for less than five days. All methane production curves in Figure 6 suggest the existence of a long tail with ~400 l/s methane flow rate and over 90% methane purity. A number of goaf holes with long production periods (7-20 and 7-MG19) also confirm that there is a large potential to increase the production period to 60-100 days without compromising methane quality or the presence of spontaneous combustion risk. In addition, as noted above, a goaf hole with longer production period also indicates that the face would be able to move further away from that hole and result in less volume of O₂ ingress.

Goaf holes drilled in the panel start-up region and completion region observed high level of CO concentration. The high CO level triggered the premature closure of goaf holes, particularly in the start-up region due to the TARP. These problematic goaf holes had relatively low methane flow rate at low methane purity. Goaf holes with low flow rate but high methane purity were also observed, such as 7-47 to 7-63, and these holes tend to have long active time. Goaf holes at the side generally have higher flow rate and higher methane purity compared with goaf holes at the tailgate side.

High CO concentration is a direct result of ventilation air migrating behind the retreating longwall and further ingress into goaf holes. Rich air (O₂) flow observed in goaf holes provides an environment for the slow coal oxidation as well as heat accumulation around the goaf hole vicinity. This leads to a delayed response of CO release and consequently observed in captured gas. The air sucked in goaf holes was most likely from ventilation air leakage while fresh air passed through the longwall face. It is reasonable to anticipate that as the face moves away from a goaf hole, the amount of ventilation air migrate to that goaf hole will be much lower. This can be validated by goaf gas profiles (Figure 10 and Figure 11), where CO/O₂ level dropped and CH₄/CO₂ level increased as the face moved 300 m away from that hole. While the reason behind the introduction of CO level trigger to switch-off the goaf holes is not known, it has certainly affected negatively on the optimum goaf hole operation to minimise the TG gas levels. Therefore, it is recommended to re-consider the usage of CO level in TARPs to switch off the goaf holes or consider the possibility to measure goaf gas temperature as an indicator of the elevated level of coal oxidation.

The current CO-based TARP resulted in the premature closure of a large number of goaf holes in the study mine. Findings from this data analyses suggest it is arguable to not simply use CO as the TARP for identifying spontaneous combustion activity to shut-off goaf holes. A number of reasons are summarised below:

- The release of CO from the coal oxidation process is a delayed response, which requires time for CO percentage to accumulate to the trigger level. Compared with CO, the consistent presence of O₂ in drained gas can be observed earlier and should also be taken into consideration regarding the potential elevated levels of coal oxidation process.
- The progressive longwall retreating causes dynamic changes to CO, O₂, CO₂ and CH₄ levels. For a goaf hole at a fixed location, even though high CO has been observed soon after the goaf hole starts production, the rapid advance of the face will soon isolate the goaf hole from the O₂ source (ventilation air) behind the longwall face. A general declining trend of CO level as the face moves away from that goaf hole can be anticipated.
- The current CO-based TARP focuses on a single threshold value rather than the trend over a consecutive period. As noted in this analysis, the accumulation of CO normally shows an upward trend given that sufficient O₂-rich environment lasts for a long period. A single record of maximum CO value may rise the concern but should not be treated as the trigger to shut-off the goaf hole. Temporary increase of CO may be caused by a slow-down or
stoppage of the longwall face, and the CO level will decline to an acceptable range once normal face production resumes.

- Once high CO has been detected in a goaf hole, it is useful to examine the CO level in its neighbouring goaf holes since CO release is found to be spatially correlated. A robust diagnosis process be placed underground using a tube-bundle and bag sample monitoring regime rather than using goaf hole gas composition to evacuate the mine for spontaneous combustion risk.

CONCLUSIONS AND RECOMMENDATIONS

This paper analysed gas drainage performance from vertical surface goaf holes in an operating mine with high specific gas emission rates (~20 m³/t). Field manual monitoring results of daily goaf gas flow rate and gas composition collected from individual goaf holes were analysed. The major challenge of implementing the full potential of goaf drainage system is the perceived risk of sponcom in a goaf using arbitrary CO trigger set points, without any scientific basis or empirical evidence. Based on the analysis of extensive dataset from the study mine, this analysis suggests that the following approaches can be taken to maximise goaf hole gas drainage performance without compromising underground sponcom risk management:

- The start-up period and completion period of a longwall panel showed a general trend of CO increase, which indicates the presence of conducive environment for coal oxidation. The deployment and operation strategy of goaf holes in these two periods should be carefully reviewed with specific reference to the use of CO trigger level.

- Optimum control of goaf gas drainage rate based on the distance between the goaf hole and face can be applied, whereby goaf holes are operated in low flow rate when they are close to the face but high flow rates when they are away from the face.

- From an explosion management perspective, the key trigger gases to be monitored are oxygen and methane rather than the use of CO as a trigger to control the goaf hole operation. It is strongly recommended that no goaf holes should be operated at methane concentrations below 30% regardless of the oxygen concentration to foolproof the probability of explosion risks. If there is a consistent presence of O₂ were observed in a number of neighbouring goaf holes, temporary shut-in of these holes should be applied as per the leading practice. Reopening of these holes can only be considered when the face is at least 300 m away from these holes, and their gas composition should be reviewed to ensure the O₂ concentration has dropped to an acceptable level.

- The analyses of data as well as empirical evidence clearly suggested that the use of CO gas as a trigger in goaf hole to identify underground sponcom activity could not be established and use of CO levels based on goaf hole gas composition for mine evacuation is flawed.

- While the investigation into the origin and introduction of CO trigger in TARP could not be established, the paper provides adequate background and the basis to eliminate or revise the use of CO triggers in current surface goaf gas management TARPs. The continued mis-use of CO as a trigger for underground sponcom activity has a detrimental effect on longwall TG gas management for explosion prevention due to early goaf hole termination despite higher levels of methane purity.

- Longwall operations must continue to deploy and monitor underground goaf environment using appropriate tube bundle monitoring and bag sample regime, real-time longwall tailgate airflow and CO monitoring for early indication of sponcom related activities.

REFERENCES
Balusu, R, 2016, Personal communications, CSIRO, Australia.


A REVIEW OF ENERGY SOURCES OF COAL BURST IN AUSTRALIAN COALMINES

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Abstract: Coal burst, which refers to the violent and catastrophic failure of coal, is a serious safety hazard for underground coalmines. Coal burst has attracted intensive research interest from mining and geological scholars. Due to the shallow mining depth, simple geological condition, advanced mining technology and reasonable geotechnical design, coal bursts has not been identified as a safety hazard in Australia coalmines as there are no documented coal bursts cases in Australia before 2014. However, more recently, the coal burst risk in Australian coalmines was highlighted by coal burst accidents in some coalmines. This paper reviewed the potential energy sources and their influence on past coal burst accidents in Australia. It is believed that the previous coal burst accidents in Australia are more likely to be the dynamic failure of highly stressed coal triggered by small-scale dynamic disturbance. Coal burst propensity index method and micro seismic monitoring technic are recommended to indicate coal burst risk before and during mining activities.

INTRODUCTION

Coal burst, which refers to the violent and catastrophic failure of coal in underground mining, has long been known as a safety hazard in coalmines of Poland, Russian, China and the U.S. (Dou et al., 2006b; Mark, 2017; Whyatt, 2008). Even after decades of extensive research made by international mining and geotechnical scholars, the increasing trend of coal burst frequency and severity appeared in these countries and has not been solved. Presumably due to the shallow mining depth, simple geological condition, advanced mining technology and reasonable geotechnical design, coal bursts has not been identified as a safety hazard in Australian coalmines as there is no documented coal burst cases in Australia before 2014. However, following the coal burst accidents that happened in some Australian coalmines, it is believed by mining researchers and engineers that Australian coalmines will face the safety hazards posed by coal bursts in the future (Yang et al., 2018a).

During the occurrence of a coal burst, massive stored elastic energy in the coal body will be rapidly released in the forms of noise, coal ejection and seismicity (Bieniawski et al., 1969). Intensive researches have been conducted to understand the energy sources of coal bursts. The increased static load resulting from high depth of cover was identified as an important energy source of coal burst by many researchers because of the positive correlation between coal burst frequency and mining depth (Dou and He, 2001). The study delivered by Zhao, et al., (2017) illustrated that major geological structures, such as faults, folds, and coal seam thickness change, can cause the stress and energy concentration. In addition, the experimental result (Vardar O et al., 2017) and theoretical analysis (Yang et al., 2018a) indicated that surrounding rock stiffness is a dominant factor of energy flow between the coal seam and surrounding rock. The influence of dynamic load on the mechanical behaviour, elastic energy ratio and burst potential of coal had been studied by many researchers as well (Li et al., 2016; Okubo et al., 2006). The mining experience of the U.S. and China found that many coal burst cases are closely related to the dynamic load caused by seismicity events (Ge, 2005). Hence,
the existence of dynamic load in coalmines can encourage the formation of coal burst by providing additional seismic energy.

To better explain the formation of coal burst, Dou, et al., (2015) proposed the static and dynamic load superposition theory to illustrate the energy sources of coal burst. As shown in Figure 1, coal burst will occur when the sum of static and dynamic load exceeds the minimum load required for coal burst formation. It also has been widely accepted by other researchers that, in most cases, the dissipated energy of coal burst is provided by the combination of static and dynamic load (Linkov, 1996; Whyatt et al, 1900). However, the study on the relationship between coal burst accident happened on 2014 and seismic event in New South Wales delivered by Ahn, et al., (2017) found no clear correlation between seismic events and mining activities, which means dynamic load, or at least seismic events, may not be an important factor for coal burst formation in Australia. This finding is not a general rule as it is just the result based on the coal bursts case that happened in 2014. Further study needs to be conducted to figure out the energy sources of coal bursts in Australia as four more coal burst accidents happened in 2016 and 2018. The aim of this paper is providing a better understanding of the energy sources of coal burst in Australian coalmines. The reminder of this paper reviews the possible energy sources of coal bursts and its potential influence in Australian coalmines the formation of coal bursts.

![Figure 1: Coal burst induced by static and dynamic load superposition (Wang et al, 2016)](image)

**STATIC LOAD**

**Mining depth**

Mining depth has been identified as an important factor for the formation of coal burst. According to the analysis of coal burst cases in Poland and China, Dou, et al., (2006b) found that the first coal burst accident in coalmines generally happened when mining depth approaches 350m and the frequency and severity of coal bursts sharply increased with mining depth changing from 350 to 600m. Some scholars found that nearly all coal burst accidents in the main coalfields of the U.S. occurred at depths greater than 300m, and most were in exceed of 400m (Mark, 2016). The contribution of mining depth to coal bursts mainly result from the increasing gravitational stress. More strain energy will be stored in coal under high gravitational stress condition (Dou and He, 2001). Besides, for coalmines in China and the U.S., hard sandstone roof seems the common geological feature for deep mining, which can further result
in a large accumulation of energy or a catastrophic dynamic load (Agapito and Goodrich, 1999; Yang et al., 2018a). The potential influence of hard roof (roof stiffness) also will be discussed in other section of this paper. The mining depth of two coalmines with coal burst accidents in Australia are around 500m (Mine Safety, 2016). Hence, the strain energy accumulation leaded by high gravitational stress plays an important role in the formation of coal burst accidents in Australia as the mining depth of the coalmines is already beyond the mining depth of the majority of burst accidents revealed by international research.

More seriously, almost all coalmines in Australia have plans for deeper mining, which means the stress environments will be more complicated and more energy will be stored in coal seams (Zhang et al., 2017).

Geological structures

It has been shown by numerous studies that the complicated geological structures caused by folds, faults and coal seam thickness variation have a noticeable influence on the coal burst occurrence (Iannacchione and Tadolini, 2016). Dou and He, et al., (2001) found that 72% of coal burst accidents in Longfeng Colliery were related to faults. The numerical study conducted by Chen, et al., (2012) found that stress will concentrate near the coal face when the coal face approaches a fault. Mark (2017) found that coal burst accidents in the U.S have close relationship with faults. Folds, which are created by compressional tectonic stress, may have high residual tectonic stress in the geological structures. Through the stress regression analysis of Huanghuayan Colliery, Jiang, et al., (2018) found that stress concentration tends to exit at the area near the syncline axis. The influence of geological structures on stress distribution is shown in Figure 2.

**Figure 2: Stress concentration caused by geological structures**

Compared with the condition of other chief coal mining countries such as China, the U.S. and Canada, most of the coalmines in Australia are in coal seams with sample geological conditions and covered by gentle and order sedimentary basins. However, evidence shows that complicated geological structures are involved in the coal burst occurrences in Australia as well. According to the investigation reports published by NSW Department of Industry, two coal burst accidents that happened in 2014 and 2016 are both in faulted zones (Bruce and Jim, 2017; Mine Safety, 2016). Besides, as shown in Fig.3, these two coal burst accidents also happened in the area with many large faults. The coal burst accidents that happened on 2
February 2018 and 17 May 2018 are also relevant to the geological problems caused by faults. The latest coal burst accidents occurred in the Bulli seam. In general, as shown in Figure 3, faults are not intense in the Bulli seam while this seam is often associated with folds and the regional geological structure of this seam is a broad syncline (Hutton, 2009). Bulli seam in the area where coal bursts occurred is under bad roof conditions caused by orthogonal joints (Brook, 2016).

Surrounding rocks stiffness

Stiffness of the surrounding rocks is one of the main factors giving rise to coal burst. Bieniawski found that rock samples are more prone to violent failure under the high stiffness loading machine. The uniaxial compression tests of samples composed of coal and rock found that most elastic energy is stored in the coal part of the compound sample and burst potential of the sample is positively related to the thickness of rock part (Dou et al., 2006a; Huang and Liu, 2013). Through theoretical analysis, Yang, et al., (2018a) found that energy will flow from high stiffness material to low stiffness material. Hence, high stiffness of surrounding rocks will enhance the energy accumulation in coal seams. In addition, as shown in Figure 4, the strength of coal tends to rapidly decrease under a high stiffness environment (Vardar et al., 2017). Generally, the high stiffness environment is related to the heavy and hard sandstone layer above the coal seam (Whyatt, 2008). Sometimes, the thickness of the sandstone layer can reach tens or even hundreds of meters (Dou and He, 2001).
As shown in Figure 5, the Branxton Formation, which generally consists of more than 400 meters thick sandstone and conglomerate units, is described as strong and massive roof above the Greta seam (Mine Safety, 2016). The existence of high stiffness roof is a potential factor that can cause massive elastic energy accumulation in Greta seam. However, the Bulli seam in Illawarra Measures, which is the coal seam mined at Southern coalfield, is under a weak and highly jointed roof. Hence, there may be no roof above Bulli seam as thick and hard as the Branxton Formation.

Figure 5: Generalized stratigraphic column for the geological Sydney Basin (Herron et al., 2018)

**DYNAMIC LOAD**

**Earthquake**

Earthquake, refers to the large-scale seismic events. If they happened in or near coalmines, they can result in a rapid accumulation of seismic energy in coal by applying dynamic stress transferred by vibration waves. Generally, an earthquake is caused by geobody instabilities related to mining activities such as fault slipping and breakage of overburden strata. In 2007, seismic event with local magnitude ML=3.9 occurred in Crandall Canyon coal mine, Utah and the seismic wave was recorded by earthquake monitoring stations operated by the United States Geological Survey (USGS), the University of Utah, and EarthScope (Ford et al., 2008). Powerful seismic events related to copper mining activities have been detected in Poland (Lizurek et al., 2015). As the largest coal producing country around the world, China has more than 80 reported mining induced earthquake records and the magnitude of most earthquake is around 3 ML (Li et al., 2007). The mine earthquakes that happened at Lander and Hector even reached magnitude 7.3 and 7.1, respectively (Gomberg et al., 2001).

After the first coal burst accidents happened in 2014, Ahn, et al., (2017) analysed the seismic events that occurred within the New South Wales mining regions from June 2006 to June 2016 and found no clear correlation between coal bursts and the past-recorded seismic events. Geoscience Australia, a preeminent geoscience organization supported by Australian government, operates a high-quality seismograph network that provides ongoing coverage for locating and recording earthquakes that occur within Australia. Using the earthquake monitoring data published by Geoscience Australia, the seismic events that occurred near coal burst spots
from March 2014 to June 2018 are drawn in Figure 6. It is clearly illustrated by the seismic data that there were no monitored seismic events near coal burst spots before and after the coal burst accidents. Hence, there was no large-scale mining induced earthquake in mining area when coal bursts were happening.

Figure 6: Seismic events occurred near coal burst spots in recent years
Micro seismicity

Micro seismicity refers to the regional small-scale seismic events which are undetectable by earthquake monitoring station due to their small-scale energy compared with earthquakes. However, for underground coalmines, the energy released by micro seismicity is an important energy source for coal burst formation. Intensive micro seismicity has been observed in most of coalmines with high burst risk in Poland, China and the U.S. (Hallo, 2012; Leśniak and Isakow, 2009; Li et al, 2018). Micro seismicity can be detected and located by specific micro seismic monitoring apparatus. Deep research has been made by many researchers on the monitoring of dynamic load and identifying high burst potential areas through micro seismic monitoring (Amitrano et al, 2010; Ge, 2005; 2010). In 2013, CSIRO established a micro seismic monitoring system at the 2014 coal burst spot to monitor the longwall weighting. The field monitoring results clearly demonstrated the effectiveness of micro seismic monitoring to infer longwall caving and weighting events (Shen et al, 2013). However, most of the micro seismic events recorded by geophones were weak.

DISCUSSIONS

The coal bursts that have occurred in Australia possess the general features of coal burst including high mining depth, complicated geological structures or massive roof strata mentioned by Poland, China and the U.S. researchers as well. However, strong dynamic load does not seem to be a factor which leads to coal burst in Australia as there are no reported mining induced seismic events or strong seismic events sensed by mining workers. Coal burst propensity index method, a widely adopted risk evaluation method of coal burst in Russian, China and the U.S., can indicate the coal burst risk from the aspect of energy accumulation and dissipation. The detailed introduction of this method can be found in literature (Yang et al, 2018b). As shown in Figure 7, the coal burst propensity index test conducted by Yang, et al., (2018b) also found that the elastic energy storage ability of Australian coal seam with high burst potential is typically high. There is no record of a coal burst happening in Australian coalmines with shallow depth, which also indicates that there is no dynamic load strong enough to trigger coal burst in these coalmines. Dou divided coal bursts into two types including strong dynamic load type (coal is under low static load but coal burst can be induced by strong dynamic load) and high static load type (coal is under high static load while small dynamic disturbance can leads to coal burst) (Dou et al, 2015). As analysis above, it is believed that most of the energy released during coal burst accidents that happened in Australia is provided by high static load. That is, the coal bursts happening in Australia are more likely to be of the high static load type. Hence, the coal burst controlling measures involving blasting operations may not be suitable for Australian coalmines as the further dynamic disturbance caused by blasting may trigger the instability of coal. For other coalmines with great depth, high stiffness strata, and complicated geological structures, the coal burst may be easily triggered by small-scale dynamic load if massive energy has been stored in the coal. The elastic energy storage ability of coal is a dominant factor of coal burst formation. Hence, the coal burst propensity index method can be an effective way to evaluate the burst risk of coalmines as this method can indicate the energy storage ability of coal (Yang et al, 2018b). Micro seismic monitoring is also a useful geotechnical tool to indicate coal burst risk by the real-time monitoring and locating of seismic wave released by small-scale dynamic load.
CONCLUSIONS

This paper reviewed the potential energy sources and their influence on coal burst formation of past coal burst accidents that occurred in Australia. Based on the analysis above, the following conclusions can be drawn:

1. High mining depth and complicated geological structures are the common features for coalmines with coal burst history in Australia. According to international experience, these factors can result in stress and strain energy concentration in coal. Hence, coal burst propensity index is recommended to be adopted as a coal burst risk evaluation method for these coalmines.

2. There is heavy and massive strata above the Greta seam while the roof of the Bulli seam is weak and poor. High stiffness roof is one of the potential factors which can cause elastic energy accumulation of the Greta seam. But the strong roof may not be a source of strong dynamic load as there is no reported seismic events related to roof weighting.

3. The coal bursts that happened in Australian coalmines are more likely to be the failure of highly stressed coal triggered by small-scale dynamic loads. In this circumstance, micro seismic monitoring should be considered as a geotechnical tool to indicate coal burst risk by real-time monitoring and locating of small-scale dynamic loads.

However, this is the preliminary conclusion with limited geological and geophysical information. A more quantitative analysis can be conducted with detailed geological mapping and sufficient micro seismic monitoring data around the coal burst sites.

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REFERENCES


Mine Safety (2016) IIR16-05 Austar Coal Burst. NSW Department of Industry.


Whyatt, J., Blake, W., Williams, T. and White, B. (1900) 60 Years of Rockbursting in the Coeur D'alene District of Northern Idaho, USA: Lessons Learned and Remaining Issues.


A CAUSATION MECHANISM FOR COAL BURSTS DURING ROADWAY DEVELOPMENT BASED ON THE MAJOR HORIZONTAL STRESS IN COAL, VERY SPECIFIC STRUCTURAL GEOLOGY CAUSING A LOCALISED LOSS OF EFFECTIVE COAL CONFINEMENT AND NEWTONS’ SECOND LAW

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ABSTRACT: This paper outlines what is considered to be a credible, first-principles, mechanistic explanation for these three current development coal burst conundrums by reference to early published coal testing work examining the significance of a lack of "constraint" to coal stability and an understanding of how very specific structural geology and other geological features can logically cause this to occur in situ, albeit on a statistically very rare basis. This basic model is examined by reference to published information pertaining to the development coal-burst that occurred at the Austar Coal Mine in New South Wales, Australia, in 2014 and from the Sunnyside District in Utah, USA.

The "cause and effect" model for development coal bursts presented also offers a meaningful explanation for the statistical improbability for what are nonetheless potentially highly-destructive events, being able to explain the statistical rarity being just as important to the credibility of the model as explaining the local conditions associated with burst events.

The model could also form the basis for a robust, risk-based approach utilising a “hierarchy of controls”, to the operational management of the development coal burst threat. Specifically, the use of pre-mining predictions for likely burst-prone and non-burst-prone areas, the use of the mine layout to avoid or at least minimise mining within burst-prone areas if appropriate, and finally the development of an operational Trigger Action Response Plan that reduces the likelihood of inadvertent roadway development into a burst-prone area without suitable safety controls already being in place.

INTRODUCTION

In 2017, one of the international authorities on coal bursts, Dr. Chris Mark, published a paper entitled “Coal Bursts that Occur During Development: A Rock Mechanics Enigma”, in which several relevant technical issues were identified, the most pertinent being:

(i) whilst development coal bursts are commonly associated with geological faults, understanding which specific faults result in burst-prone development mining conditions and why, remains undefined.

(ii) conventional wisdom in relation to strong roof and floor geology of the coal seam might be limiting, based on certain burst examples in Colorado.

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(iii) development coal bursts can occur without the local ground stresses being substantially elevated from their in situ levels.

The key to understanding the problem of coal bursts during roadway development is in explaining why, at a particular location, does many tonnes of coal in an otherwise stable mining environment, suddenly and without warning become unstable and rapidly accelerate horizontally into a mine roadway without the obvious influence of excessive gas pressures, to the point that the event itself can be heard many hundreds of metres away in the mine. As an industry, the answer to this question appears to remain largely incomplete.

Based on published case histories and information, coal bursts that occur during roadway development without the influence of multi-seam stress interactions, statistically at least, appear to be the underground coal-mining equivalent of a lightning strike. However, at least with lightning strikes, even if their exact timing and location cannot be predicted, the general atmospheric conditions under which they are most likely to occur are well-understood so that effective safety measures can be enacted. Mark 2017 suggested that as recently as mid-2017, the general conditions under which development coal bursts were most likely to occur, remained unresolved.

This paper attempts to address the following:

- provide a possible causal explanation linked to the specific stress conditions under which development bursts can occur, and hence define where they are most likely to occur,
- link this explanation to the statistical rarity of such events more generally, and
- briefly consider how the explanation can lead to a structured development of coal burst management approach based around a Hierarchy of Controls.

The paper is a first-principles-based review of public-domain information from the coal burst at the Austar Mine in 2014, along with published information from the Sunnyside District in Utah.

Definitions

A general review of the literature relating to coal bursts quickly reveals an obvious lack of consistent terminology across bursts, bumps and gas outbursts, it being acknowledged by many that some of the recorded historical coal “bursts” in the USA were more likely to have been gas outbursts. Today a myriad of descriptive terms are seemingly in use in an attempt to classify many different types of events, such as “rock burst”, “strain burst”, “pressure burst”, “coal burst”, “pillar burst”, “shakedown”, “pressure bump/bounce”, “coal bump” and “pillar bump”. The problem with this type of classification is that it is based on the observed manifestation of an event, rather than the source of the energy and the mechanism by which it was released. A good example is found in Gale 2018 whereby it is stated that “a coal burst is defined as a rapid expulsion of coal (and potentially gas) from the boundary of the roadway. The volume of a burst can be variable, but volumes above 10-50m³ are noted and cause significant disruption to operations”. Such a definition, whilst being accurate in a descriptive sense, is actually not helpful when attempting to understand causal mechanisms, therefore another classification method is judged to be required.

Figure 1 contains a basic Venn diagram approach to defining different types of “high energy release” events based on the source of the released energy. Bursts are taken to be those events whereby the energy released is strain energy from within the coal seam, bumps are those related to strain energy release from either the overburden above or floor below the coal seam, and outbursts are primarily driven by gas pressures within the coal seam. This is not to then say that an overburden bump cannot result in a violent expulsion of coal into the mine workings, simply that the energy source involved is not from within the coal seam.
Figure 1: Suggested classification of high energy release events in underground coal mining based on energy source

Figure 1 also recognises that some events can be combinations of two or more energy sources, although from an investigative perspective it may be wise to first understand events based on single energy sources and mechanisms, rather than increasing the complexity by attempting to understand multiple-energy source events.

Figure 2 provides a schematic illustration of how an overburden or floor “bump” might manifest from an energy release perspective, due to horizontal stress-induced shear slip within a thick, massive strata unit along a mid-angled fault plane. There are other possible source mechanisms for overburden bumps, but a simple energy source and release mechanism representation such as that in Figure 2, allows first-pass bump hazard-ranking to be undertaken, based on (a) the presence or absence of thick, massive strata units in proximity to the coal seam, (b) the presence or absence of mid-angled fault planes that extend through such massive units, and (c) the major horizontal stress being aligned sub-perpendicular to the fault plane.

(a) roof bump due to shear slip along the fault plane in massive strata

(b) floor bump due to shear slip along the fault plane in massive strata

Figure 2: Overburden and floor bump causal mechanisms due to horizontal stress-induced shear slip along mid-angled fault planes

Whilst it is not the focus of this paper, if one accepts the event definitions contained within Figure 1, it is immediately apparent that some events that have been classified by others as coal bursts or pillar bursts, are almost certainly bumps, and vice versa. Without the correct
classification of individual events according to the energy source and release mechanism, the search for understanding as to cause and effect is likely to remain elusive.

**DIFFICULTIES IN DEFINING A CREDIBLE DEVELOPMENT COAL BURST CAUSATION MODEL**

Determining “cause and effect” is fundamental to developing improved engineering solutions to problems, yet despite 100 years of coal bursts occurring during roadway development (albeit on a statistically rare basis), according to Mark 2017 the problem remains an “enigma”, this being “something that is difficult to understand”, rather than one that cannot be explained.

If one examines other geotechnical problems in coal mining such as pillar design and roadway roof control as examples, current understanding and associated control practices commonly emanates from combined industry experience as encapsulated in both empirical databases that have been statistically analysed, and detailed monitoring studies of changes in relevant conditions during mining. It is through the resulting insights that mechanistic cause and effect models have been developed that allow improved hazard identification and engineering solutions to be applied in current mining operations.

With development coal bursts, there are (a) so few events that have been well-documented that an industry database approach would be flawed from the outset, and (b) as a very rapid “lightning strike” type event whose timing and location were unknown prior to the event, no targeted monitoring data typically exists to help define exactly what occurred and in what sequence during the event. Therefore, from an analysis perspective, one is forced to take a “first-principles” approach, the credibility of which then must be judged against whatever case history details are available, the 2014 Austar incident being the only Australian example on record not accompanied by a significant release of gas as far as can be determined.

**Loss of confinement hypothesis**

Babcock and Bickel 1984 investigated the idea that coal bursts could be directly caused by “a loss of constraint”, which would then allow other ground stresses in the coal itself to cause a coal burst. They proved the concept in the laboratory by rapidly removing the lateral confining pressure on coal samples that were already highly stressed in the perpendicular direction, finding that violent failures occurred in 15 different types of coal material. They also showed that with lateral confinement being removed, the coal failure type and severity was dictated by the contact conditions between the coal cubes and the steel platens of the test machine, this typically being defined by zero cohesion (i.e. steel onto coal) and minimised friction as per standard rock testing specimen preparation.

Having tested coal from 15 mines in 11 seams in 6 US states with 13 being made to burst when lateral confinement was removed, Babcock and Bickel 1984 concluded as follows:

“we believe that many, if not most, coals can be made to burst given the necessary conditions of stress and constraint. In cases where the strength is largely produced by constraint, the sudden loss of this constraint can produce bursting”, and

“strain energy can produce bursts without the help of gas pressure”

Their work demonstrated that most coals would burst due to the release of internal strain energy if stressed in one direction without adequate constraint in the others, thereby taking the emphasis away from the strength of the coal and more towards the ground stresses acting within the coal seam and variations thereof. This is the focus of this paper.

**Identifying the development coal burst energy source**
If a development coal burst is taken as being caused by an energy release from within the coal seam itself, then there are only four possible energy sources:

(i) gas pressure
(ii) major horizontal stress
(iii) minor horizontal stress
(iv) vertical stress

If it is further accepted that an event driven by gas pressure is a gas outburst as defined in Figure 1, then the event source mechanism for a development coal burst (ignoring multiple-energy source events for now) must inevitably focus on the three principal ground stresses, with the confining or constraining influence of two needing to be overcome by the third, as postulated by the general Babcock and Bickel 1984 model.

Recent work reported by Gale 2018 includes the statement that “Computer modelling indicates that, once confinement and cohesive strength develops in the ribside, the resistive energy becomes much larger, and significantly greater energy is required to create a burst within the confined material”. This is generally consistent with the findings of Babcock and Bickel 1984 albeit that the focus of Gale 2018 is to identify and justify the source of the “greater energy” within the coal seam that is required to cause a coal burst in a confined state. The focus of this paper is to understand how confinement of the coal can locally be lost in two of the three principal stress directions, thereby allowing normal strain energy within the coal seam to drive a coal burst in the remaining direction.

Taking this one stage further, it is instructive to consider the stored energy due to the major horizontal stress within a section of a coal seam, and the resultant acceleration of the coal should that energy become unstable and be released in the manner of an unloading spring. It is noted that strata behaving as a spring under load is also discussed in Gale 2018, whereby he states that “it can be visualised by viewing the rock as a spring, which is compressed by the in situ stresses. The stored energy is the amount required to have compressed the strata (spring) to the in situ state”.

If a coal mass accelerates horizontally when it bursts, it will be assumed as a starting assumption that it is the major horizontal stress that is the primary event driver. For a major horizontal stress of 5 MPa acting with a 3 m high by 3 m long and 3 m wide (i.e. 27 m³) block of coal weighing some 38 tonnes (i.e. 27 m³ x 1.4 = 38 tonnes), the stored horizontal force = stress x area = 5 x 100 tonnes/m² x 3 m x 3 m = 4,500 tonnes of horizontal compressive force or stored energy.

If that stored energy was to become unstable, based on Newton’s Second Law, the resultant acceleration = force/mass = 4,500/38 = 118 m/s² = 11.8 g. This acceleration results in a velocity for 38 tonnes of coal = 22 m/s = 80 km/hr at a distance moved of only 2 m. The mechanics and destructive potential of 38 tonnes of coal moving at 80 km/hr requires no further comment herein, suffice to state that this confirms the assertion of Babcock and Bickel (1984) that a normal level of ground stress in a coal seam is more than sufficient to cause a very destructive coal burst if it becomes unstable due to a loss of constraint. How such a state can come about in situ prior to mining is therefore the focus of the remainder of this paper.

The normal or typical state of In Situ Stress

If one accepts for the moment that development coal bursts are both (i) driven by the major horizontal stress in the coal seam becoming unstable due to a loss of effective constraint, and (ii) statistically very rare, then the normal or typical state of the in situ stresses must be such that coal bursts on development cannot possibly occur. Therefore, the starting point for this
discussion is to consider the normal or typical state of *in situ* stress in coal mining and whether it potentially allows development coal bursts to occur or not.

Referring to Figure 3, the pre-mining 3D stress state in coal measures has three principal components, one vertical and two typically being horizontal. Based on the previous discussion with regard to development burst causation, for the major horizontal stress to become “unstable”, it needs to be able to overcome the combined constraining or stabilising influence of both the vertical stress and minor horizontal stress. Therefore, the sources of the minor horizontal and the vertical stresses need to be defined, so that how they may be overcome by the major horizontal stress can be considered further.

![Figure 3: Schematic illustration of assumed pre-mining principal stresses](image)

The *in situ* vertical stress is well established as being caused by the weight of the overburden, as illustrated in Figure 4. No other explanation is required in this regard.

![Figure 4: International vertical stress measurement summary (Hoek and Brown 1980)](image)

The link between the major horizontal stress in coal mining and plate tectonic effects is well established and does not need repeating herein. However, the very strong relationship that is almost always found between the measured magnitudes of the major and minor horizontal stresses is less well known, as outlined in more detail in Colwell and Frith 2012. Figure 5 shows
an example of such a relationship from mine site stress measurements, the finding being that
as the major horizontal stress increases in magnitude, so does the minor horizontal stress,
typically being between 50 to 60% of the magnitude of the major horizontal stress. The reasons
behind this commonly found relationship are irrelevant to the objectives of this paper, suffice to
state that the minor horizontal stress typically acts to stabilise the major horizontal stress, rather
than allow it to become critically unstable.

![Figure 5: Sample stress measurement data showing a strong linear correlation
between the major and minor horizontal stresses](image)

With the vertical stress being driven by cover depth and the major and minor horizontal stresses
tending to increase and decrease linearly with each other, this offers a credible explanation for
the statistical rarity of development coal bursts, as the typical or normal in situ ground stresses
in coal mining do not obviously conform with the Babcock and Bickel 1984 hypothesis whereby
one principal stress becomes unstable due to a loss of effective constraint from the other two.

This then raises two key questions:

1. mechanistically how, on a very rare basis, the constraining influence of both the minor
horizontal and vertical stress can be lost or overcome by the major horizontal stress,
thereby allowing the major horizontal stress to be the energy source for a development coal
burst, and

2. is there any credible evidence indicating that such conditions were present at known
development coal burst sites?

These two questions will now be considered further.

**Loss of the minor horizontal stress with the major horizontal stress being maintained**

The one obvious scenario whereby one principal stress can be reduced in magnitude back to
zero, and the other maintained and even intensified, is the stress re-distribution that occurs in
2D around an excavation, as illustrated in Figure 6 for a circular excavation under hydrostatic
stress conditions. If one considers Figure 6 in plan, rather than section, so that the two stresses
being analysed are horizontal, it is evident that at the boundary of the excavation, the tangential
stress (i.e. that acting parallel to the excavation boundary) is intensified as a result of the
excavation being formed, whereas the radial stress (i.e. that acting perpendicular or normal to
the excavation boundary) drops to zero at the boundary. The question that therefore follows is
whether there is a credible scenario that allows such an excavation or void to form via natural
processes prior to mining, so that a local modification to the in situ pre-mining horizontal

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*Figure 5: Sample stress measurement data showing a strong linear correlation
between the major and minor horizontal stresses*
stresses is induced, consistent with the minor horizontal stress being lost and the major horizontal stress maintained?

![Figure 6: 2D elastic stress redistribution around a circular excavation under hydrostatic stress conditions](image)

Figure 6: 2D elastic stress redistribution around a circular excavation under hydrostatic stress conditions

Figure 7 is taken from an underground coal mine in South Africa and demonstrates that open voids via open defects (e.g. joints, cleats or faults) can, and do indeed develop pre-mining, the horizontal stress acting perpendicular to an open defect inevitably being zero at the boundary of the defect. However, for the development coal burst causation model to be credible, a mechanistic explanation for the formation of substantial pre-mining voids within an otherwise bi-axial compressive horizontal stress environment needs to be developed.

![Figure 7: Example of an open vertical joint in an underground coal mine](image)

Figure 7: Example of an open vertical joint in an underground coal mine

Figure 8 is taken from Hatherly et al 1993 and illustrates the formation of new fractures (as marked in red) at the tail end of a section of horizontal shear movement along a major fault plane driven by the major horizontal stress (NB these new fractures are termed as “wing cracks” in structural geology parlance). It is noted that Hatherly et al 1993 was in part based on structural mapping conducted at Ellalong Colliery, which is part of the larger Austar Mine complex.
The following quotation is taken from Hatherly et al 1993: (emphases added by authors):

“Strains near re-activated fractures and new fracture propagation. Re-activation of pre-existing fractures creates additional strains which leads to the development of new fractures through the TENSILE FAILURE of intact material. These fractures tend to curve into an orientation sub-parallel to the major compressive stress”.

The excerpt from Hatherly et al 1993 describes the exact characteristics required by the development coal burst model for a local modification to the major and minor horizontal stresses within the coal seam, namely that the major horizontal stress is maintained (parallel with the tensile/open fracture), whilst the minor horizontal stress is inevitably eliminated (perpendicular or across the tensile/open fracture) as shown in Figure 9, if the aperture or width of the tensile fracture is sufficient to accommodate the necessary strata relaxation.

Figure 8: Strains near re-activated fractures and new fracture propagation (Hatherly et al 1993)
Loss of the constraint from the vertical stress to the major horizontal stress

Whilst a causation mechanism has been identified that can explain the local loss of the minor horizontal stress whilst the major horizontal stress is maintained or even intensified, the same cannot be applied to the vertical stress due to it being driven by the weight of overburden, which cannot be so easily relieved, if at all. Therefore, another mechanism and set of circumstances is required to explain how the constraint to the major horizontal stress from the vertical stress can be overcome.

The solution is straightforward and is found by considering the resistance to horizontal shear movement along a horizontal plane, as illustrated in Figure 10. For a large mass of coal to be rapidly ejected sideways out into a mine roadway, the horizontal driving stress needs to overcome the horizontal shear resistance within the coal seam, which will logically be at a minimum along any discrete and continuous horizontal defects, planes or low strength beds.

Figure 9: Zoomed-in area of wing-crack development showing inferred local horizontal stresses

Figure 10: Confining influence of the vertical stress on horizontal shear movement
Depending upon the cohesion and friction acting along such a plane, it can readily be demonstrated that the major horizontal stress in the coal seam cannot always be effectively constrained by the vertical stress in isolation. For example, for a zero-cohesion horizontal plane having an Angle of Friction of say 10° (which as a comparison represents a slickensided, planar bedding surface), results in only 0.176 MPa of horizontal shear resistance for every 1 MPa of vertical stress. Even in a coal seam containing relatively low tectonic horizontal stresses due to the low Young’s Modulus of coal, such a level of horizontal shear resistance from the vertical stress is insufficient to fully constrain and control the major horizontal stress.

**SPECIFIC CIRCUMSTANCES AT THE AUSTAR INCIDENT SITE**

Various details of the Austar incident have been made publicly available in NSW Department of Industry 2015, NSW Department of Industry 2016 and Hebblewhite and Galvin 2016. Two geological circumstances of the event are believed to be of direct relevance to causation under the hypothetical coal burst mechanism described herein as will now be explained.

Figure 11 shows the reported structural geology of the incident site with key features being highlighted, namely the main “Quorrobolong” fault zone some distance outbye the incident site (shaded in green) and a structure or series of structures projected to be just inbye the incident site (highlighted by the blue dashed line) that are oriented substantially differently to the main fault zone, but project back to the main fault zone just to the south of MGA9 (see Figure 12). The difference in the alignments of these two different structure sets is directly comparable to that shown in Figures 8 and 9 relating to the development of tensile wing cracks due to variations in horizontal stress-driven shear movement along a pre-existing fracture.

![Figure 11: Plan showing layout and some geological features of mg a9 (NSW Department of Industry 2015)](image-url)
If it were to be the case that the structure or structures indicated by the blue dashed line in Figure 11 were in fact tensile wing-cracks developed from horizontal shear movement along the main fault plane, they would provide for the necessary pre-existing strata “void”, with the potential ability to locally eliminate the minor horizontal stress, whilst maintaining if not more likely intensifying the major horizontal stress, as illustrated in Figure 9.

Figure 12 shows a photo of the geological structure where it was intersected at the inbye end of the adjacent A Heading (see Figure 11), and under one interpretation it could conceivably be associated with an open void that has subsequently been filled with extraneous material over geological time. It is accepted that the complete nature of the structure(s) just inbye the current headings in MGA9 cannot be defined from Figure 12. However, it is impossible to ignore this structural zone given its different orientation as compared to the main fault zone, what this may signify in terms of its genesis, and the potential for it to have contained a distinct strata void at some point in geological history.

Taking this suggestion a stage further, it is hypothesised that if the major and minor horizontal stresses were locally modified by a substantial strata void within the structural zone, it should result in stark differences in coal rib conditions either side of any nearby roadways, as illustrated schematically in Figure 13. In this regard the following are noted:

(i) roadway rib conditions in proximity to the burst site are described in NSW Department of Industry 2016 as follows:

“Mining conditions in B Heading at the time included some spall of the right hand rib, below the Dosco Band (see Figure 6, for an image of the Dosco Band parting within the coal seam). However the left hand rib was standing straight.”
(ii) Post-event geotechnical mapping of the general area as reported in NSW Department of Industry 2016 (see Figure 14), indicates that in the adjacent A Heading, the left-hand rib was mapped as Condition Green which is defined as < 0.3 m of rib spall, the small photo of the left-hand rib included in Figure 14 showing a rib that is “standing straight”. In contrast, the right-hand-rib is mapped as Condition Red which is defined as > 1 m of rib spall and described as “very friable and sugary inbye A2”.

(iii) Figure 15 from NSW Department of Industry 2015 shows the right-hand rib in A Heading, which provides a clear indication of the significant levels of rib fracture and spalling, as compared to the left-hand rib that was described as “standing straight”.

The conclusion drawn from the available evidence is that inbye the main fault zone, both A Heading and B Heading exhibited quite unusual rib condition variations between the left-hand and right-hand sides of the roadway. One possible explanation for this is an intensification of the major horizontal stress and substantial reduction of the minor horizontal stress within the coal seam, as indicated in Figure 13.
Figure 14: Post-incident geological mapping of area (NSW Department of Industry 2016)
In terms of whether a very weak horizontal plane of weakness was present at the Austar burst site and so acted to allow uncontrolled horizontal shear slip of the coal by reducing the confining influence of the vertical stress below critical levels, the presence of the Dosco Band at the top of the coal section that burst is clearly evident in Figures 16 and 17. This raises the question as to whether the Dosco Band in this location was likely to be characterised as being of zero cohesion and low friction?
Figure 17: Looking into the burst cavity from the position of the continuous miner
(NSW Department of Industry 2016)

NSW Department of Industry 2015 contains the following statement – “The smooth and dominant shear surface presented by the Dosco Band within the Greta Seam, which appears to have acted as a dynamic shear failure plane once some form of triggered loading (or unloading) event occurred”. Hebblewhite and Galvin 2016 provide a more detailed description – “the upper bound of the burst cavity is clearly visible (Fig. 6b) as a very smooth, flat bedding plane within the seam known as the “Dosco Band”. Rib coal above the Dosco band has not displaced at all, whereas all the coal beneath it is part of the burst. The exposed surface of the Dosco Band showed signs of horizontal shearing activity, with a quite distinctive reddish-brown dust coating on much of the surface. Newman (2002) and others have reported similar evidence of reddish-brown pulverised coal particles at burst sites”. These two independent descriptions of the exposed Dosco Band at the Austar incident site clearly confirm it as being a planar or flat surface with little or no cohesive strength and minimal friction.

Another possible source of a planar, zero cohesion, low friction horizontal plane of weakness within a coal seam that could theoretically act in a similar manner to that of the Dosco Band in a coal burst, is the unconformable contact that commonly exists between the top of a coal seam and base of a massive strata unit such as a sandstone. The significance of such a contact is clearly evident in Figure 18 from a US coal burst site, this offering a credible explanation for the commonly observed presence of a strong sandstone roof at coal burst sites, which has potentially resulted in many researchers inadvertently placing the significance of the presence of sandstone on its high strength or modulus, rather than the very specific nature of the contact between the coal and the sandstone.
FEATURES OF THE SUNNYSIDE DISTRICT, UTAH

According to Mark 2017, the Sunnyside District in Utah was the location of the first recorded development coal burst in the USA in 1915, subsequent to which many similar development burst events were recorded.

Figure 19 shows the regional structural geology of the Sunnyside District, which is dominated by the Sunnyside Fault Zone. The Sunnyside Fault Zone is described in Osterwald et al 1993 as under-going horizontal shear-slip due to the action of horizontal stress, as evidenced by the bending of railway lines on the surface following large regional bump events.
Figure 19: Major structural geology of the sunnyside mine area (Osterwald et al 1993)

More specifically in relation to the phenomenon of development coal bursts, several features appear to be consistent with either the structural geology of the Austar incident site or the development burst hypothesis presented in this paper more generally, they include:
(i) the difference in alignment between the main fault zone in Figure 20 and what appears to be another mapped structure extending out from it, with the associated “bump area” being located between the two structures. This is broadly similar to the situation from Austar shown in Figure 11.

(ii) The fact that the main fault zone shown in Figure 20 has one side characterised as being associated with bumps and the other as not being associated with bumps. The application of the wing crack causation model shown in Figure 8 would lead to the inevitable conclusion that a major fault zone would only be coal burst prone on one side at any given location.

![Figure 20: Mapping showing relation of faulting to bumping and non-bumping areas in part of the sunnyside No.2 mine (Osterwald et al 1993)](image)

The geological mapping in Figure 21 shows three relevant features in close proximity to the main fault zone, namely one side of the heading being heavily spalled (marked by the red shaded area) as compared to the other side (as suggested from Figure 13 and demonstrated via information from Austar), mapped geological structures at a significantly different alignment to the main fault zone (marked by the green shaded area) and an area described as “broken coal, thrown out from left rib” (marked by blue shaded area) on the side of the heading with the more stable general rib conditions.

The mapped structural geology and varying roadway conditions from the development coal burst-prone mine’s in the Sunnyside District show similar general features to that from the development coal burst location at Austar. Further, the indication that one side of a major fault was burst prone and the other not burst prone, adds further credibility to the wing-crack model, as shear-slip movement driving the formation of new tensile fractures (wing-cracks) would logically only occur on one or other side of a major fault at any given location along its length.
CAUSAL MECHANISM SUMMARY

In the process of developing and illustrating a credible causation model for understanding development coal bursts, questions posed within Mark 2017 were used as a starting point, particularly related to the following two comments:

"coal mines have developed across many faults in Utah and elsewhere. What was so unique about the Sunnyside fault that it contributed to so many powerful bursts over so many years, many in the same place and well outbye any active mining?"
and why is the area so burst prone, when the coal seams are encased in relatively soft rock?

Furthermore, the work attempted to address the following broader questions pertaining to development coal bursts:

- What is the energy source?
- Is there a mechanistic link between the manifestation of a development coal burst and its severity?
- Can the ground stresses and/or geotechnical conditions which (a) create the energy source and (b) allow it to become unstable (and so is released in an uncontrolled manner) be defined?
- How can such geotechnical conditions come about in an underground mine?
- Is there any evidence that such geotechnical conditions were present at Austar in proximity to the burst site (based on public domain information) and/or other known development burst–prone mines (e.g. Sunnyside in Utah)?
- Does the development coal burst model provide a plausible explanation for the statistical rarity and improbability of development coal bursts in general terms?

Addressing these questions in their entirety, the following summary points are made:

1. The energy source for a development coal burst can demonstrably be simply the major horizontal stress within the coal seam (even at relatively low magnitudes), as evidenced by the application of Newton’s Second Law, providing that the constraining influence of the minor horizontal stress and vertical stress are insufficient.

2. The minor horizontal stress can locally be substantially reduced to as low as zero, with the major horizontal stress being maintained or even intensified via the development of dilated tensile fractures known as wing-cracks at the tail end of horizontal shear-slip along a major fault plane, this being well-established in structural geology.

3. The constraining influence of the vertical stress can be eliminated via the presence of a planar, zero cohesion and low friction horizontal plane, either in the form of an unconformable contact at the top or bottom of a coal seam, or a discrete stone or clay bed within the coal seam that has either been re-worked by horizontal shearing effects over geological time or is naturally very weak and friable. Interestingly, this is consistent with the comments of Babcock and Bickel 1984 in that they observed that once lateral constraint was removed from a coal sample. the style of failure was directly linked to the contact conditions between the coal sample and steel platens of the testing machine.

4. The required geological conditions for a development coal burst, as outlined in points 2. and 3., were seemingly present in direct proximity to the Austar incident site, and more generally in the Sunnyside District in Utah.

5. The horizontal stress conditions and the necessary structural geology under which such stress conditions can form within an otherwise bi-axial compressive horizontal stress environment, combined with the need for a specific horizontal plane of weakness to substantially reduce the constraint provided by the vertical stress, offer a credible explanation for the statistical improbability of development coal bursts across industry in general terms.

The proposed development coal burst causation model outlined in this paper provides what are considered to be credible answers to the various questions listed, remembering that it is founded on well-established physics and structural geology, in combination with known local conditions from the only well-documented development coal burst in Australia, and general
geological conditions within a mining district in the US that was demonstrably prone to
development coal bursts.

**IMPLICATIONS TO COAL BURST MANAGEMENT**

If a Hierarchy of Controls approach is applied to the issue of development coal burst
management in mining operations, it inevitably results in the following requirements, in priority
order:

1. To eliminate exposure to the hazard if possible.
2. To develop suitable engineering controls that minimise both the exposure to and severity
   of the hazard if 1. is not possible.
3. To develop effective administrative control measures to prevent the inadvertent exposure
   of persons to the hazard if 1. and 2. above fail.

With these three statements in mind, the following comments are made for general industry
consideration:

a) major geological structures that have undergone a substantial local relative change in
   horizontal shear-slip magnitude along the structure (i.e. strike-slip motion) under the action
   of the major horizontal stress (i.e. a very specific local structural influence), might be
   identified pre-mining from geotechnical borehole exploration data, including variations in
   the direction and magnitude of the major horizontal stress, as can be inferred from borehole
   breakout.

b) the critical “danger” area from a development coal burst perspective, is proximity to the “tail
   end” of substantial horizontal shear slip along a fault plane where new tensile fractures
   (“wing-cracks”) can form and substantially modify the horizontal stress environment.

c) it is possible that high horizontal stress, coal-burst prone zones might be identified and
delineated by longhole drilling. In this regard, Figure 22 shows the pre-development
longhole drilling at the outbye end of MGA9, firstly through the Quorrobolong Fault Zone
and then inbye towards the what would eventually be the coal-burst site and the geological
structure just slightly further inbye. The reason that longhole drilling was stopped as shown
was reported “bogging” as stated in Figure 22. The founding reference (NSW Department
of Industry 2016) does not offer an explanation as to what this term means in reality,
however the comment is made that “Directional longhole drilling was undertaken from the
300 Mains in advance of Maingate A9 development, to establish the presence and nature
of some of these projected geological features, as well as to inform on coal seam continuity
and gradients ahead of Maingate A9 development. Figure 13 shows the pattern of holes
that were drilled in the vicinity. While these holes clearly penetrated the cluster of faults that
crossed Maingate A9 between 1 and 2 cut-through, they appeared to have all stopped short
of the structures that were encountered beyond 2 cut-through in A Heading, and that were
projected to lie just beyond the face of B Heading and the dog-leg heading at the time of
the incident”. Without digressing in detail, it is stated that the actual mine workings appear
contain no obvious major geological structure (see Figure 14) that would explain the
“bogging” and ceasing of longhole drilling, an alternative potential explanation being
intensified horizontal stresses within the coal seam causing excessive coal breakout and
hole collapse at this location. The close alignment of the limit of long hole drilling and the
inferred “wing-crack” structure just inbye the burst site is intriguing, as is the mapped
occurrence of floor heave in A Heading commencing just inbye the limit of longhole drilling
(see Figure 14).
Figure 22: Longhole drilling undertaken ahead of maingate A9 development (NSW Department of Industry 2016)

d) zero cohesion and low friction horizontal planes within the working section (i.e. clay bands, re-worked stone bands, sandstone seam roof/unconformity) which substantially limit the constraint generated by the vertical stress, should be identifiable from surface borehole information.

e) items (a) to (c) potentially allow some form of credible pre-mining hazard ranking for development coal burst prone and non-prone areas.

f) the mine layout could be engineered to eliminate exposure to identified burst-prone areas, or at least minimise both the exposure to and severity of the hazard.

g) the development burst causation model leads to the identification of some obvious visible and audible TARP triggers that could be used in operations to identify the onset of development coal burst-prone areas, including stark variations in roadway rib conditions and the identification of certain types of geological structures.

The mechanical engineering and isolation control measures that may allow roadway development in coal-burst prone conditions, such as remote mining, guarding on the CM and/or out-of-seam drivage, are well beyond the scope of this paper. Therefore, no comment as to the likely effectiveness of such measures has been considered or is provided.

CONCLUSIONS

The paper has outlined suggested generic definitions for different types of “high energy” release events that can occur in the underground coal mining based on the location and type of the energy source, rather than the characteristics of the manifestation in the mine workings. This is considered to be fundamental to improving the understanding for each event type and hence, the ability to more reliably predict and manage the associated safety and business risks.

A causation model specifically for development coal bursts has been outlined, founded on the loss of constraint hypothesis that was first postulated by Babcock and Bickel 1984 from their laboratory testing studies. Using published information in regards to development coal bursts at the Austar Mine in Australia and from the Sunnyside District in Utah, USA, a model linking the local structural geology to (a) the loss of the minor horizontal stress and (b) significantly limiting the constraint offered by the vertical stress has been developed, the resulting inevitable uncontrolled release of the major horizontal stress in the coal seam being shown to be more
than sufficient to generate a violent expulsion of coal by reference to Newton’s Second Law of Motion.

The causation model further addresses two key question raised in a seminal paper on development coal bursts from as recently as 2017, namely faulting system characteristics that are likely to be development burst prone, and how can coal bursts occur within a sequence of otherwise soft measures. Furthermore, it also offers a credible explanation for the statistical rarity of development coal bursts in more general terms, this being due to the improbability of all of the required elements of the causation model coming together at the same location within a mine roadway.

The causation model allows more targeted hazard ranking prior to mining as well as inputting into operational management of the development coal burst hazard. In particular, if the proposed causation model is essentially correct, credible pre-mining hazard ranking for development coal burst prone areas should be achievable using commonly available exploration information such as horizontal stress directions and severity from borehole breakout analyses, and horizontal planes of weakness from lithological logs. Further, there is credible evidence that longhole drilling ahead of roadway development is able to delineate development coal burst-prone zones via the inability to drill into them.

Ultimately, industry will benefit from being able to apply a more targeted Hierarchy of Controls approach to the development coal burst problem, which due to the very high associated safety and business consequences will benefit from being focused on both prediction ahead of mining and detecting the hazard before mine roadways are developed into development burst-prone areas without suitable mitigatory controls in place.

REFERENCES


A SCIENTIFIC APPROACH TO QUANTIFYING THE EFFICIENCY AND EFFICACY OF DUST CURTAINS ON A SANDVIK MB650 CONTINUOUS MINER AT KESTREL COLLIERY WHILST MINING A FULL FACE OF ROOF STONE DURING AN OVERCAST CONSTRUCTION

Brian Plush¹, Bradley Watson, Clinton Day, Timothy Gooch, Charlie Spence

ABSTRACT: With the correct identification and continued increase in CWP and related occupational lung disease in the Australian coal mining industry since May 2015, the industry’s focus has been directed at mining operations achieving statutory respirable dust level compliance to AS2985. The majority of dust control techniques currently installed and operational in Australian coal mines have been developed in the USA, UK and other western countries and their application is more suited to low to medium coal seam heights up to 3m. The Australian mining experience has indicated that the efficiency of some of the existing respirable dust control methods reduce significantly in thick coal seams, under high production environments and when mining roof stone. As the current trend in the industry is to substantially increase production levels, there is an urgent need for detailed investigation of various dust control options and development of appropriate dust management strategies based on quantifying the efficiency and efficacy of installed controls to mitigate respirable dust from the working environment. This paper details the approach taken to quantify the efficiency and efficacy of installed face curtains for respirable dust mitigation on a Sandvik MB650 continuous miner whilst mining a full face of roof stone cutting an overcast in the mains at Kestrel Colliery. Results of the project have shown that the installed face curtains are not suitable as a dust mitigation control.

INTRODUCTION

Coal Workers Pneumoconiosis (CWP) or Black Lung, is an insidious disease that is totally preventable. Questions relating to the validity and subsequent suitability of the current respirable dust sampling methodologies utilised in Australia have come under significant scrutiny, as they have failed to mitigate the exposure risk to Coal Mine Workers (CMW’s) to as low as reasonably achievable, evidenced by the confirmed 87 cases of Coal Workers Pneumoconiosis (CWP or black lung) and other Mine Dust Lung Diseases (MDLD) since 1984 (DNRM website accessed 22/11/2018)

It is well understood that the measurement of respirable dust at the source of generation is a difficult task and that the current process for measuring exposure levels for respirable dust has significant limitations (Plush, et al 2012). Recognised Standard 14 acknowledges that assessing and managing the risk of respirable dust exposure is complex and may require the use of specific risk assessment techniques (Recognised Standard 14). Further, section 3 (2) notes that in order to ensure the risk of black lung, or other disease created from exposure to

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respirable dust is at an acceptable level, the sampling regime must be appropriate, statistically robust and professional judgement undertaken by a competent person at data interpretation (Recognised Standard 14).

The limitation to Recognised Standard 14 and AS2985 – 2004, *Workplace atmospheres—Method for sampling and gravimetric determination of respirable dust*, lays in the fact that the existing statutory testing regime only monitors the exposure level of the worker to the hazard. If an exceedance occurs, the monitoring process does not indicate why the exposure occurred, where the source of the exposure originated from, nor if the currently installed engineering controls are working effectively (Plush et.al; 2012). This has created an industry strategy of removing the person from the hazard through task rotation or operator positioning, which is an administrative control under the Hierarchy of Controls for risk management. According to Recognised Standard 15, sections 5.2 and 7.8, an administrative control such as task rotation and operator positioning requires monitoring and review, does not control dust generation and requires compliance to s.89 of the Regulation.

Dust measurement not dust monitoring is required to eliminate dust at the source of generation. Simply monitoring how much dust is in the air, as previously noted, has limited value. Removing the person from the hazard does nothing to remove the hazard from the air, and only provides a temporary administrative control, the second lowest point on the Hierarchy of Controls. Benchmarks are required to understand how much dust is produced at independent sources of dust generation, then measured again with installed controls operating to determine the efficiency of the installed control at mitigating the risk. Elimination is the highest point of the Hierarchy of Controls and is achievable if the efficiency and efficacy of installed controls is measured and quantified. Exposure level testing will never provide a basis for dust elimination.

Section 5.3.2.2 of Recognised Standard 15 relates to the requirement that, when equipment is selected for purchase, where maintenance is undertaken on equipment and during equipment overhauls, “….all equipment purchases shall be considered in terms of the hierarchy of controls..” and that “…equipment purchase and specification requires input from persons with the relevant expertise in the area of engineering control of dust generation…” (Recognised Standard 15).

Further, Appendix B of Recognised Standard 15 sets a requirement that “Equipment specifications shall include requirements for commissioning plans that confirm the supplied equipment meets the specification requirements for dust control, and *include a plan to establish baseline effectiveness of the dust control equipment*…..”

In relation to the Sandvik MB650 used for mains and gateroad development at Kestrel Colliery, the above requirements from the equipment supplier have not been provided. Kestrel identified that not much progress over the last year had been achieved in reducing the risk profile for respirable dust throughout the underground and surface operations of the mine. Kestrel has a Dust Committee and individuals assigned to different areas but without a baseline the pit does not know where to focus or which controls are the most effective and those which should be focused on to lower the risk to as low as reasonably achievable.

**TESTING METHODOLOGY TO QUANTIFY THE EFFICIENCY AND EFFICACY OF MB650 FRONT CURTAINS**

Planned engineering control efficiency testing was undertaken in the Mains area of Kestrel Colliery where recent personal sampling results have recorded exceedances in silica dust during cutting high drivage excavations involving grading into and out of the roof stone. The area tested was “A” heading 59-60ct as shown in the flight plan in Figure 1.
Data collection involved cutting 1m of advancement for the baseline establishment, ie, with no controls operable excluding pick sprays for frictional ignition compliance, and another meter with each of the installed engineering controls operable and working as designed. The 1m cut was undertaken as 2 x 500 mm cuts due to the amount of stone in the face. Testing was continued until the completion of the second cut which took the overall advancement to 1m. At the completion of the first 1m cut, the pumps and heads were turned off and removed to allow the operators back on the miner to install necessary roof controls. During this process, all water was turned back on and any float dust hosed down. After roof controls were installed, the pumps and new set of heads were reinstalled at the same locations, the water to the miner was turned on and the second set of testing commenced. This process was repeated every meter until the completion of the tests.

Table 1 below summarises how each of the tests was performed. The collected data quantified the efficiencies of the installed engineering controls at respirable dust mitigation based on the establishment of a baseline dust production at each of the sources sampled.

<table>
<thead>
<tr>
<th>Test</th>
<th>Pick Sprays</th>
<th>Face Curtain</th>
<th>Side Curtains</th>
<th>Sprays</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>On</td>
<td>Retracted</td>
<td>Retracted</td>
<td>Off</td>
</tr>
<tr>
<td>Test 2</td>
<td>On</td>
<td>Operable</td>
<td>Retracted</td>
<td>Off</td>
</tr>
<tr>
<td>Test 3</td>
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<td>On</td>
</tr>
<tr>
<td>Test 4</td>
<td>On</td>
<td>Operable</td>
<td>Operable</td>
<td>Off</td>
</tr>
</tbody>
</table>

**QUANTIFYING ENGINEERING CONTROL EFFICIENCIES**

The current testing regime in Australia, AS2985, provides the mine tested with a single figure for respirable dust exposure levels during a production shift. This figure only provides information relating to the exposure levels of the person sampled, relative to the 300 mm
breathing zone described in AS2985 and does not provide any feedback on the effectiveness of installed engineering controls or any other information that would allow the mine site to implement improvements in mitigation procedures should a non-compliance, or failure to Statutory regulations occur.

The testing methodology for this efficiency testing project utilises dust loads as opposed to exposure levels. The objective of this sampling methodology is to identify dust loads at independent sources of dust generation on the continuous miner and quantify the efficiency of installed controls for the mitigation of produced dust. This data will be used to create a benchmark or signature in relation to dust loads from different sources of generation on the continuous miner. Once this signature is established, quantifiable testing can be undertaken on installed controls to ensure maximum efficiency in removing respirable dust is achieved.

The samples are collected as per AS2985 Gravimetric sampling process; however, the collected data is analysed as a raw weight taken for the benchmark with no controls operating and as a raw weight taken with controls operating. The difference between the two raw weights is the efficiency of the installed control at mitigating respirable dust.

An important aspect for the scientific robustness and repeatability of this sampling process is that the raw weights are analysed compared to tonnes cut as opposed to sampling time as a Time Weighted Average, (TWA), ensuring that the efficiency sampling of installed controls can be confidently repeated.

Figure 2 below shows the face curtains and throat curtains in normal operating conditions. The throat curtains are down, and the face curtains are down.

Figure 2: Face and throat curtains

DETERMINE SAMPLER LOCATION

In each location, as discussed above, a pump and respirable head was used to sample dust loads produced during the cutting cycle to establish a baseline dust production and the efficiency of the installed engineering controls.

Pumps and respirable heads were placed on all continuous miners tested as detailed in Figure 3. These positions are:
• Pump 1 was placed on the left-hand inside side bolter controls to measure dust that is bypassing the front curtains and rolling back over the miner.
• Pump 2 was placed on the left-hand side outside bolter to measure dust that is bypassing the front curtains and rolling back over the miner.
• Pump 3 was placed on the left-hand side of the bolting cassette to measure the amount of dust generated through the throat and out onto the miner platform.
• Pump 4 was placed on the left-hand side rear hand rail to measure what dust is being brought back in with the ventilation from loading the shuttle car.
• Pump 5 was placed on the right-hand inside side bolter controls to measure dust that is bypassing the front curtains and rolling back over the miner.
• Pump 6 was placed on the right-hand side outside bolter to measure dust that is bypassing the front curtains and rolling back over the miner.
• Pump 7 was placed on the right-hand side bolting cassette to measure the amount of dust generated through the throat and out onto the miner platform.
• Pump 8 was placed on the right-hand side rear hand rail to measure what dust is being brought back in with the ventilation from loading the shuttle car.

![Figure 3: Pump and head placement on MB650](image)

EFFICIENCY TESTING PROCEDURE

The first set of tests were undertaken without the dust suppression sprays operating and other installed dust controls inoperable. This allowed the measurement of the dust load produced during the cutting cycle.

Note:

• Pick sprays remained on for frictional ignition control.
• Miner driver wore a Clean Space during the testing process.
• All other coal mine workers were removed from the hazard to a location of known fresh air.
The controls were made inoperable or turned off as follows:

• The front curtains were rolled up as high as possible and down as low as possible to allow the produced dust to bypass them. This was achieved by chain blocking the curtains up as high as possible and then securing them as required for the testing. This process was repeated to pull down the curtains to the roof. Figure 4 shows a photo of the retracted curtains.

• The side curtains will be rolled up and cable tied securely on either side securing them as required for the testing.

• All sprays, excluding pick sprays, were turned off. It was anticipated that the other sprays would be disconnected at the back of the sprays to allow the water to flow through the cooling circuit and dump to ground.

Figure 4: Retracted curtains

The second set of tests were undertaken as follows:

• The front curtains were replaced to their original position and made operable as per their design.

• The side curtains remained rolled up and cable tied securely on either side securing them as required for the testing.

• All sprays, excluding pick sprays, were turned off. It was anticipated that the other sprays would be disconnected at the back of the sprays to allow the water to flow through the cooling circuit and dump to ground.

The third set of tests were undertaken as follows:

• The front curtains were replaced to their original position and made operable as per their design.

• The side curtains were rolled down to their original position and made operable as per their design.
• All sprays, excluding pick sprays, were turned off. It was anticipated that the other sprays would be disconnected at the back of the sprays to allow the water to flow through the cooling circuit and dump to ground.

The fourth set of tests was undertaken as follows:
• The front curtains were replaced in their original position and made operable as per their design.
• The side curtains were rolled down to their original position and made operable as per their design.
• All sprays were turned back on and made operable as per their design.

RESULTS

During the establishment of the benchmark, it was found that the installed curtains were creating more dust and forcing it to migrate down the platform when they were down as designed. The visible and measured respirable dust was significantly higher with the curtains down as designed compared to when the curtains were rolled up and inoperable. Figure 5 shows the respirable dust being forced around the curtains on the RHS of the continuous Miner (CM) and almost to the rear of the operator’s platform with the curtains down as designed. The produced respirable dust remained against the face when the curtains were rolled up.

![RHS Curtains Down](image1)

![RHS Curtains Up](image2)

Figure 5: RHS curtains down and curtains up dust migration

Figure 6 shows the respirable dust being forced around the curtains on the LHS of the CM and down the platform with the curtains down as designed. The produced respirable dust remained against the face when the curtains were rolled up.

![LHS Curtains Down](image3)

![LHS Curtains Up](image4)

Figure 6: LHS curtains down and curtains up dust migration

The obtained results indicate that with the front curtains down and operating as designed, throat curtains off, and all sprays operating as designed, significantly more respirable dust is produced and migrates down the operator’s platform of the CM. Significant respirable dust mitigation was
measured with the front curtains rolled up and inoperable, quantifying that the front curtains are not suitable as a dust mitigation control.

Table 2 shows the results achieved by removing the front curtains and leaving the throat curtains and sprays operating as designed.

**Table 2: Results achieved by removing the front curtains**

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Location On CM</th>
<th>Sprays On</th>
<th>Sprays Off</th>
<th>Efficiency</th>
</tr>
</thead>
<tbody>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>LHS Inside Bolter</td>
<td>0.0678</td>
<td>0.0008</td>
<td>-99%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>LHS Outside Bolter</td>
<td>0.0589</td>
<td>0.0016</td>
<td>-97%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>LHS Mid Platform</td>
<td>0.0054</td>
<td>0.0008</td>
<td>-85%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>RHS Inside Bolter</td>
<td>0.1572</td>
<td>0.0047</td>
<td>-97%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>RHS Outside Bolter</td>
<td>0.1869</td>
<td>0.0024</td>
<td>-99%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>RHS Mid Platform</td>
<td>0.0062</td>
<td>0.0034</td>
<td>-45%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>Miner Driver</td>
<td>0.0186</td>
<td>0.0013</td>
<td>-93%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>LHS Rear Platform</td>
<td>0.0042</td>
<td>0.0001</td>
<td>-98%</td>
</tr>
<tr>
<td>22-Jun</td>
<td>Mains</td>
<td>RHS Rear Platform</td>
<td>0.0058</td>
<td>0.0002</td>
<td>-97%</td>
</tr>
</tbody>
</table>

Figure 7 is a graphical representation of the obtained results.

**Figure 7: Graph of the respirable dust reduction achieved by removing the front curtains**

Figure 8 shows the reduction in respirable dust at different positions on the CM.

**Figure 8 – Respirable dust reduction achieved by removing the front curtains**
CONCLUSIONS

Based on the above results, it can be quantified that the front curtains installed on the MB650 Continuous Miners are not suitable for dust control. Further, mining a full face of roof stone on CM3215 at Kestrel Colliery has occurred under the following parameters:

- Removal of front curtains to allow face pressure to hold the produced respirable dust against the face allowing the whale's mouth to remove it;
- Throat curtains installed and operating as designed;
- All installed sprays operating as designed;
- The miner driver is to be no closer to the face than at the operators' screen;
- All other personnel are to be back behind the miner driver or off the miner during the cutting cycle, with the exception of dust technicians to take videos and photos during the sampling process;
- Ventilation behind the miner should be no lower than 0.4m/s and 0.5m/s on either side of the miner mid platform;

Continued sampling will be undertaken to further quantify the efficiency and efficacy of the installed throat curtains, throat sprays, tail sprays, apron sprays and Temporary Roof Support (TRS) sprays which will create a best practice engineering control setup for cutting stone with continuous miners, allowing cutting and bolting simultaneously, whilst ensuring the respirable dust risk potential is as low as reasonably achievable.

REFERENCES


APPLICATION OF COMPUTER AIDED ENGINEERING TECHNIQUES IN COMBINATION WITH HIGH-ENERGY DUST SUPPRESSION TECHNOLOGY FOR THE HANDLING OF COAL

Jon Roberts¹, Michael Hopkins², Peter Wypych³, Vitold Ronda⁴

ABSTRACT: The coal industry faces significant challenges in the control of dust to meet emissions regulations and goals as well as ensuring sustainable operations. This paper describes some of the different techniques and innovative technologies that are being developed and implemented to improve the suppression of airborne dust, specifically through the use of numerical modelling in combination with high-energy micro-mist sprays. In the handling of coal, ROM bin loading, transfer points, and discharge to stockpiles are identified as common and significant sources of dust, specific industry examples are presented. To tackle these areas, the utilisation of CFD and CFD-DEM simulation modelling is identified and described as a key enabling technology for an improvement in dust suppression technology both from a level of understanding of the source and dynamics of these emissions, and for the development of new systems that can be used in the coal industry. CFD-DEM modelling is outlined as a method for analysis of dust and air flow, whilst CFD modelling is shown to be effective for modelling spray dispersion and dynamics under varying conditions. New high-energy micro-mist technology is also outlined as a key element in developing high-efficiency dust suppression systems. The application of these technologies is shown as applied to industrial problems at various Australian mines with data presented demonstrating the reduction in dust emissions that can be achieved.

INTRODUCTION

In the coal industry, dust emissions are an increasingly troublesome issue that has seen very little improvement achieved for many years. In Australia, industry emissions of particulate matter less than 10 microns in size has increased from 530 million kilograms in 2009/2010 to 920 million kilograms in 2013/2014, representing a significant and increasing problem (Australian Government - Department of the Environment, 2014). Issues associated with excess dust emissions include health implications, environmental pollution, material loss, and equipment deterioration due to the adverse operating environment. Worker morale and productivity can also be negatively affected by excess workplace dust, and of course, there is the important need to comply with increasingly stringent regulations primarily from a pollution and health perspective. These issues vary with dust properties and concentration, which is directly related to the quantity of material handled and the control methods implemented. Water sprays designed to wet material as a way of limiting dust release is one of the primary methods used for dust control, however, the effectiveness of this method is limited and varies from application to application. Many of these systems also suffer from high consumption of valuable clean water. Improved design methods in combination with high energy micro mist nozzles will be presented in this paper as a means of developing much higher efficiency dust control

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systems with lower rates of water consumption and decreased costs compared to the water spray systems commonly in use today.

In the coal industry, there are numerous handling operations which result in the liberation of hazardous dust particles, these include but are not limited to dump pockets, conveyor transfer points, and stockpile stacker conveyors. The variability associated with these operations due to weather conditions such as wind and rain as well as the variability that can occur with material itself, such as moisture content and tonnage, all combine to make it difficult to develop reliable dust control system for all conditions. To achieve a notable improvement in the control of dust in these areas it is necessary that new technologies be developed, and improved design techniques established. Research conducted at the University of Wollongong has identified two enabling technologies that can result in this improvement; the application of high energy micro mist sprays and the use of Computer Aided Engineering (CAE) techniques, specifically Computational Fluid Dynamics (CFD) and Discrete Element Method (DEM). The benefits and factors effecting the use of high-energy micro-mist sprays has been outlined in a number of previous papers (Roberts & Wypych, 2017), (Roberts & Wypych, 2018), (Roberts, et al., 2018).

The use of CAE techniques such as CFD, DEM, and coupled CFD-DEM for the prediction of air, dust and mist flow in industrial applications has seen considerable interest of late. A validated model of spray simulation using CFD has been developed specifically for dust suppression purposes (Roberts, et al., 2018) and the application of this model presents significant benefits for improving the prediction of dust suppression systems in industry applications. A number of studies have also be completed recently utilising coupled CFD-DEM techniques to predict air and dust flows from bulk material handling operations (Grubler, et al., 2018), (Hopkins, et al., 2018), (Schulz, et al., 2018). In this paper the application of these technologies together will be discussed and successful projects benefiting from their implementation will be presented.

**NUMERICAL MODELLING**

In many materials handling operations fine dust particles are liberated from the ore stream by displaced air as the material falls or as it impacts with a bin or hopper, the entrained air (laden with fine particles) is often displaced undesirably, resulting in a dispersion of particles in the surrounding areas. The use of DEM coupled with CFD presents the opportunity to model these processes so that the dispersion and flow of air and dust can be predicted. Accurate predictions of these processes will subsequently allow for implementation of more effective dust control measures such as dust suppression sprays or dust extraction systems. Examples of the analysis that can be conducted are shown in Figure 1. In the examples shown in Figure 1, opportunities to improve the processes or ways to implement dust control techniques are immediately evident. Firstly, for the rotary car dumper it is evident that a significant portion of the airflow is forced to the rear side of the hoppers, indicating that a control system should be placed in this region to capture the entrained dust particles. Both dust extraction or dust suppression could be considered here, however, based on the air velocity present (>5 m/s) it would difficult to implement an extraction system that could influence the flow of this air effectively. In this case it is therefore likely that the most suitable control measure would be a dust suppression system using high energy atomising sprays that can both deal with the momentum of the dust cloud and has a sufficiently dense mist to capture the dust particles present. A similar analysis can be undertaken for the truck dump process, in this case the air velocities are somewhat lower in the range of 3-4 m/s and the size and location of most dump pockets such as this make them well suited to dust suppression systems providing full coverage and resistance to the dust flow can be achieved.
In addition to understanding the air and dust flow generated by materials handling activities, numerical modelling of the sprays used for dust suppression can also be pursued and as such the resulting flow dynamics as a combination of the bulk material, airflow, and mist can be studied. This allows for a detailed analysis to be completed during the design process resulting in a more effective system. A technique using ANSYS Fluent and the discrete phase model allows for the fine mist produced by dust suppression sprays to be simulated in a computationally efficient manner. The discrete phase model uses Lagrangian trajectory tracking coupled with a continuous Eulerian phase to model droplets not as free surfaces but as discrete particles moving through the air with drag forces applied per the particle properties and specified drag laws. This technique reduces the need for a very fine mesh and simplifies the model significantly, in turn reducing the computational expense. This method has been shown to accurately predict the deflection and dispersion of sprays in both laboratory and industry conditions. Figure 2 shows an image of an actual spray captured in UOW’s mist deflection test rig, and a spray simulated using ANSYS fluent.
Figure 3 demonstrates how a simulation model can be used to understand the flow of mist under varying operating conditions, here the figure is showing how the model predicts the flow of mist for a specific nozzle operated at different pressures. In this case the angle of the spray allows for deflection to be limited, however, as the water pressure increases the amount of mist stripping off the spray cone reduces resulting in more mist impacting the desired region. The use of simulation modelling in this way is particularly important in ensuring that the droplet size distribution and droplet momentum is adequate such that ventilation and/or dust cloud momentum will not adversely affect the performance of any dust suppression system that is installed.

APPLICATIONS

It was identified at the outset of this paper that many coal handling operations result in problematic conditions that could benefit from improved dust suppression strategies. It is considered that the application of high energy micro-mist technology in combination with the CAE design approach already described could achieve this. The application of these technologies has the ability to result in a reduction in dust emissions, a reduction in water consumption, reduced design effort, and as such an overall reduction in the costs associated with design and operation of the system.

Utilising this design process, a project was recently completed at a QLD mine having significant issues with dust at the discharge from their rom bin and the transfers at their primary sizer and secondary crusher. An analysis of local airflows utilising CFD-DEM and seasonal weather data allowed for a system to be developed utilising appropriately sized nozzles to adequately capture the dust whilst not being adversely affected by local airflows. The worst case scenario was found when the dump pocket was empty which resulted in large amounts of dust laden air being displaced, with a significant portion of this escaping through the opening for the feeder; this is shown in Figure 4, on the left the opening is clean and empty before the material is dumped, on the right a dust cloud entirely fills the opening and escapes into the air.
To suppress the dust escaping at this location, firstly the velocity of the airflow being generated was predicted using coupled CFD-DEM; this is shown in Figure 5, the maximum velocity was approximately 3.5-4 m/s under the expected conditions, this velocity was used for calculation of the correct size sprays for the application based on spray simulations, and mist deflection data previously presented (Roberts, et al., 2018). A portion of the installed system is shown in Figure 6; the major dust generating source was from the opening for the feeder and the sprays applied here are shown on the right, dust generated at this point was also carried along the chain feeder and hence sprays were installed along the feeder to capture this, finally the image shows sprays directed into the transfer point to capture dust as it generated by the impact of material onto the sizer. Figure 7 shows dust concentrations measured concurrently with system not running and subsequently with the system turned on at full pressure, both measurements were completed under worst case scenario conditions with a full load dumped into an empty hopper. The average dust concentrations over the periods measured were 10.5 mg/m³ without the system and 2.01 mg/m³ with the system on, peak dust concentration reduced from 52.8 mg/m³ to 5.4 mg/m³.

![Figure 5: Estimation of velocity of air escaping feeder opening](image)

![Sprays at opening for feeder](image)  ![Sprays along chain feeder](image)

*Figure 6: Installed dust suppression system at exit of hopper, along chain feeder and at transfer from chain feeder to primary sizer*
CONCLUSIONS

This paper has outlined the issues associated with excess dust emissions with a specific focus on troublesome areas in the coal industry. High-energy micro-mist nozzles and CAE modelling techniques have been identified as enabling technologies for improving the control and capture of airborne dust. Current systems utilising low concentration coarse droplet sprays or low-velocity air atomising sprays do not provide the dust capture performance required in the adverse conditions present in the coal industry. CFD-DEM modelling has been shown to be valuable as a means of predicting air flow generated during coal handling operations, and CFD modelling of sprays can be used to predict their dispersion. It is evident from the data that was presented from a recent project that a significant improvement in the control of dust in the coal industry can be achieved through the implementation of new high energy micro mist nozzles in combination with these CAE techniques.

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REFERENCES


Figure 7: Dust concentration measured with and without the dust suppression system operating


THE AUSTRALIAN NATIONAL FACILITY FOR PHYSICAL BLAST SIMULATION

Alex Remennikov¹, Brian Uy², Edward Chan³, David Ritzel⁴

ABSTRACT: The National Facility for Physical Blast Simulation (NFPBS) has been established at a site north of the University of Wollongong, New South Wales, Australia. This facility is designed for systematic experimental studies of blast wave propagation and loading regimes, blasts damage to elements of civilian and military infrastructure, blast injury protection and other important blast related areas of research. The simulator is a state-of-the-art design having a test section of 1.5 x 2 m with dual-mode Driver capable of operating with compressed gas or gaseous explosive. Using an oxy-acetylene gas mix as Driver, blast simulations of 350 kPa incident level will be possible; peak levels and durations will be adjustable to 30 ms by Driver settings and adjustable distance to the test section. The simulator will be capable of a range of blast-test configurations including full-reflection wall targets, diffraction model targets, as well as behind-wall and blast-ingress scenarios. The NFPBS is based on the ‘Advanced Blast Simulator’ (ABS) concept. Various ABS designs have been adopted by several universities and government laboratories in the US and Canada pursuing blast-effects studies. Preliminary results from the NFPBS will be presented for both compressed gas and gas detonation modes of blast wave simulations.

INTRODUCTION

The past two decades have seen a significant increase in the number of terrorist attacks on embassies, commercial centres, government structures, industrial facilities, and residential buildings. From a structural standpoint, these attacks have highlighted the vulnerability of existing civilian infrastructure to the dynamic effects of high pressure, short duration blast loading. Civilian, government and military organizations have been addressing these vulnerabilities by developing new blast resistant design guidelines and retrofit procedures to mitigate blast hazards. The current state of blast resistant design methods is based largely on empirical observations of actual explosive testing. However, due to the dangerous, expensive and uncontrolled variables of experimental blast research, the body of experimental blast data is very limited and many aspects of the blast response of structures remain unknown. A proper blast simulator facility is required to allow systematic, highly controlled blast experiments at much lower cost, greater safety and higher fidelity than field trials. A large-scale blast simulator is equivalent in importance in blast protection research as the wind tunnel is to aerodynamics research particularly since many aspects of material failure can only be investigated using full-scale structural elements.

This paper describes the recently commissioned, National Facility for Physical Blast Simulation (NFPBS) in Australia for systematic experimental studies and development of high-performance blast protection technologies. Experimental test capabilities are the foundation for any research program in blast vulnerability and remain the ultimate method for validating blast protection technologies. Experimental capabilities generally fall into two categories: free-field trials and blast simulator facilities. For systematic experimental studies of blast loading, damage, and personal injury, field trials are exceedingly expensive and inefficient (e.g. depend on weather conditions). Blast Simulators are shock tubes specially designed to simulate the

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distinctive shock wave profiles produced by free-field explosions. The NFPBS overcomes the challenges associated with live explosive testing such as very high cost, safety, efficiency and repeatability of test results; more extensive and sophisticated instrumentation can also be applied in a laboratory setting with better controls on the test-target setup. The facility will be utilised for routine high-quality blast experiments to develop concepts of protecting infrastructure, from individual components such as windows, doors, columns, plates and walls, to system models such as bridges, dams, tunnels and buildings, and to models of city or urban environment.

The NFPBS is the result of direct collaboration between eight Australian universities, University of Sydney, University of Wollongong, the University of Western Sydney, the University of Western Australia, the University of Newcastle, the University of Melbourne, Queensland University of Technology, University of Technology Sydney and the Defence Science and Technology Group (DSTG) of the Australian Department of Defence. In 2013, this group of universities and DSTG proposed to develop and establish the NFPBS facility, which would be of high significance to blast-structure-interactions research for universities, government and industry in Australia.

The Advanced Blast Simulator design (Ritzel 2015) selected for the NFPBS facility is based on the concept of intrinsically replicating the wave-dynamics of actual free-field explosive blast including generation of an entropy gradient and a ‘true’ negative phase with secondary shock. Variants of standard shock tubes have a very limited capacity for blast-wave simulation. The primary components of an advanced blast simulator design include a Driver section followed by a specially shaped Transition Section which continues to geometrically expand then smoothly re-converge the flow; the tailored shockwave then enters the Test Section where experiments would be conducted. The Driver operates either in dual-mode using compressed gas or gaseous detonation dependent on the required pressure/impulse range. A special End-wave Eliminator (EWE) device is set at the end of the Test Section in order to eliminate reflected rarefactions or shocks affecting the Test Section as well as mitigating noise and gas efflux into the lab space.

DESCRIPTION OF THE NFPBS ADVANCED BLAST SIMULATOR

The NFPBS Advanced Blast Simulator (ABS) (Figure 1) is a state-of-the-art design capable of generating a shock wave that replicates the wave-dynamics of an actual free-field explosive blast. This includes reproduction of the negative phase (i.e. pressure dipping below ambient) and a secondary shock which follows sometime after the initial shock wave. This section briefly describes the different subassemblies and components which make up the simulator. An overview of these different sections is illustrated in a schematic in Figure 2.
Figure 2: Schematic of NFPBS Advanced Blast Simulator (ABS)

**DRIVER SECTION**

The Driver has a divergent wedge-shaped profile (of opposite of convergent, for increasing in cross-section continuously), and can operate either in Compressed Gas (CG) or Gaseous Detonation (GD) mode, depending on the requirement. Generally, CG mode produces shock waves with a more pronounced and adjustable negative phase with corresponding strong secondary shock while GD mode produces much stronger blast simulations with a weak negative phase.

![Figure 3: Compressed gas Driver mode: a) Location of membrane (i.e. diaphragm station); b) Clamping of a frangible membrane across the opening of the Driver to create a gas tight barrier](image-url)
In CG mode the ABS is operated in a similar manner to a conventional shock tube: a gas, typically air, nitrogen, or helium, is introduced into the Driver, raising the pressure above ambient. A frangible membrane (Figure 3) is used to separate the high pressure contained in the Driver from the ambient pressure in the driven section downstream. Upon reaching the desired pressure in the Driver, the membrane is ruptured quickly to allow high pressure gas to expand into the ambient pressure contained downstream. The release of the elevated pressure gas acts as a “piston” rapidly compressing the ambient air at the interface of the high pressure/low pressure gas volumes creating a propagating shock wave. The characteristic “Friedlander” blast wave shape is created by the expansion of the gas out of the divergent Driver and through the initial divergent Transition Section; once formed, the wave is smoothly re-converged into the Test Section.

In GD mode, the geometry of the Driver and downstream simulator sections remains identical with the compressed gas Driver mode. However, the elevated pressure region within the Driver is instantaneously created by detonation of combustible gas mixed with air and/or oxygen. Typical combustible gases include acetylene ($C_2H_2$) and ethylene ($C_2H_4$). The resultant shock wave propagates downstream in the simulator containment volume in the same fashion as compressed gas mode previously described. GD mode is capable of generating much higher shock levels than CG mode and has the operational advantage of not requiring the setup of a frangible diaphragm.

The gas delivery system (Figure 4) is designed to be operated from a safe distance, made possible by control valves that can be remotely operated from a control room. The decanted volume of combustible gas is precisely metered using a water filled U-tube manometer open to the atmosphere which has proven to be simple, accurate, and highly reliable. The water fill provides a gas tight seal preventing escape of the gas and provides the means for establishing fill system pressure to ensure positive flow of the metered gas into the simulator. The standpipe also serves as the primary pressure relief for the gas delivery system by limiting the maximum water column to less than 4m hydrostatic pressure.

**DRIVEN SECTION (TRANSITION SECTION AND TEST SECTION)**

Downstream of the divergent-area Driver Section, the connecting Transition Section continues to expand then smoothly and steadily re-converges the flow as a planar wave entering the constant cross-section geometry of the Test Section. The blast wave's rate of expansion is
smoothly reduced to zero by the time the blast wave arrives at the Test Section. The length of the Transition Section is set to provide sufficient run out for perturbations in the initial blast to dissipate before reaching the Test Section while minimising the rate of change of the wall curvature.

Unlike a conventional shock tube, a fully formed blast-wave is generated from the Driver. Therefore it is possible to locate targets at various distances from the first Transition Section as required for particular exposure levels similar to free-field blast where closer standoff gives stronger shock level with shorter duration. The NFPBS ABS is designed to be modular, that is, the Test Section is comprised of three segments which can be configured as required to increase or reduce the testing standoff distance (Figure 5).

The Transition Section incorporates a set of louvers in the top panel optional venting for reflected shocks propagating upstream from reflective targets which would otherwise be reflected again from the closed end of the Driver to propagate downstream and interfere with experiments (Figure 6). The louvers can also be used to reduce the blast wave duration for diffraction targets. The louvers can be selectively fixed shut by blocking-plate assemblies or allowed to open in a controlled manner by the force of the detonation. The extent and speed of venting is controlled by the adjustable mass of the louvers as well as recoil restraints if required.

![Figure 5: Modular design allows for adjustment of standoff distances: a) Reduced standoff distance; b) Full standoff distance](image)

![Figure 6: Flip-up louvers to allow for controlled venting of reflecting shock waves reflecting back from reflective targets](image)
REACTION HOUSING

The Reaction Housing at the end of the Test Section has three potential roles dependent on the experiment objective. For studies of loading and damage to diffraction targets set up in the Test Section, it is necessary to mitigate waves reflecting from the end of the Test Section once the primary blast wave has passed. For this role, the Reaction Housing serves as an ‘End-Wave Eliminator’ (EWE) and is configured with an internal shock diffuser having the form of a porous wedge. The porosity of the wedge can be adjusted to optimise its effectiveness in dissipating waves passing into the volume of the Reaction Housing which serves as a ‘dump tank’ (Figure 7a). The Reaction Housing has sufficient volume to dissipate the incoming shockwave as well as mitigate noise and gas efflux into the laboratory space.

![Figure 7: Mounting of reflective targets: a) Reaction Housing with a reflective target in pre-testing position; b) Test specimen mounted on Reaction Frame](image)

In its second role, the upstream opening is surrounded by a drilled heavy flange (reaction flange) serving as the mounting surface for reflective targets such as walls or doors (Figure 7b). The Reaction Housing is constructed of heavily reinforced steel and weighs approximately 15500kg; it is mounted on heavy casters and is free to roll on a track. Target reaction loads are coupled to the housing by the reaction flange and that energy is dissipated by the recoil.

The use of a massive Reaction Housing that has rigid mounting frame as boundary condition for target walls but is free to move globally is novel. Most test facilities assessing blast loads to walls ‘pretend’ they have a fixed and non-responsive inertial mounting frame. In reality, all frames under these loads will respond and transfer momentum. In fact, useful information is lost by not registering the momentum transferred to the surrounding structure from a target wall which is a factor highly relevant to the real-world problem of these wall/door components within larger buildings. One concept for building protection is to have deliberate wall-fail pathways that are least damaging for the global structure and personnel. For test facilities with a fixed and non-responsive inertial mounting frame, vibration and shock load would be ultimately passed to the foundation housing, which is both problematic for operations and introduces an ill-defined loss in the response analysis. Alternatively, it is not uncommon to directly fix a wall-mounting frame directly to the end of the Test Section (e.g. University of Ottawa) which is doubly problematic: the transferred load will jolt and often damage or slightly shift the entire simulator as well as not ensuring a true inertial boundary condition for the target wall.

In its third role, the Reaction Housing can be configured for studies of ‘behind wall’ and blast ingress effects including debris-throw as specified in GSA Test Protocol GSA-TS01-2003 (GSA
2003) for the evaluation of blast-resistant glazing for example. In this capacity the Reaction Housing volume can be fitted with special instrumentation and high-speed video.

**PRELIMINARY RESULTS**

The dual-mode Driver allows a wide performance range in which gaseous-detonation mode is generally used for target studies requiring strong blast with relatively weak negative phase while compressed-gas mode provides moderate to low shock levels with an adjustable negative phase. However, the use of shaped inserts for the Driver and Transition will allow tailoring of blast-wave profiles in both modes in addition to the controls for blast-wave duration described previously.

The initial phase of commissioning tests was intended to refine the operational procedures for GD and CG modes, qualify the data-acquisition system, provide data on the baseline GD waveform development down the simulator, as well as allow measurements of radiated noise to the surroundings. Figure 8 shows records obtained at the end of the first Transition and at the start of the Test Section for the GD Driver charged to about 1/3 and 1/2 of its capacity respectively. The records show excellent shot-to-shot reproducibility and high-fidelity simulation of free-field explosive blast waveforms. Minor aberrations seen in the records are largely smoothed-out by the middle of the Test Section. Figure 9 shows the record for reflected blast loading on a target door mounted to the front of the Reaction Housing again showing excellent simulation of reflected blast loading.

Qualification of the CG Driver mode of operation is in its earliest stages, and testing to date has been limited to preliminary low-level tests as shown in Figure 10. Using compressed air in the Driver at 260 kPa yields a blast simulation of about 35 kPa overpressure and 15ms positive duration at the start of the Test Section; the Driver is capable of 1.5 MPa. The waveforms demonstrate the enhanced negative phase possible with this mode of Driver operation which can be moderated by increasing the concentration of helium in the Driver gas. As with GD mode, excellent reproducibility is demonstrated.

![Figure 8: [Upper] Waveform generated using 0.071 m³ oxy-acetylene GD Driver showing results for two tests overlaid; [Lower] waveform generated using 0.14 m³ oxy-acetylene GD Driver showing two test results overlaid](image-url)
Figure 9: Reflected blast loading on a target door mounted to the front of the Reaction Housing for 0.14 m³ oxy-acetylene GD Driver

Figure 10: Waveform generated using 260 kPa compressed-air CG Driver showing two test results overlaid

REFERENCES

APPLICATION OF PLASTIC FUNNEL IN BLAST HOLE TO IMPROVE BLASTING EFFICIENCY OF OPENCAST COAL MINE AT WEST BOKARO

Ashutosh Bhaskar¹, Alok Kumar Baranwal², Praveen Ranjan³, Tapan Kumar Jena⁴, Mayank Shekhar⁵ and Debaprasad Chakraborty⁶

ABSTRACT: Blasting being one of the key activities of mining, its efficiency in terms of lower explosives consumption, improved rock fragmentation, decreased fly-rock, reduced noise and vibration level is very much desired for an effective mining operation which can be achieved by maximizing the utilization of explosive energy in the blast hole. Use of ‘reverse plastic funnel’ into the blast hole is one of the techniques for more utilization of explosives energy to improve blasting efficiency. The reverse plastic funnel is placed between explosive and stemming column in the blast hole which eliminates the contamination of explosive from drill cuttings (used for stemming), thus increases the Velocity of Detonation (VoD) of the explosive. Also, the conic shape of funnel creates a ‘Wedge effect’ guiding more of the explosive energy into the rock rather than upward out of the blast hole which helps in utilizing more explosive energy for rock breakage and reducing fly rock generation. In order to establish the benefit, trials were carried out in OB (overburden) benches of opencast coal mine at West Bokaro. In-hole VoD is measured by using Micro Trap VoD Recorder. It was found that the in-hole Velocity of Detonation (VoD) of the explosive is more in blast hole having funnel which means more strength of explosive. It was also observed that the fly rocks generation is negligible from blast holes in which funnels are placed.

INTRODUCTION

Blasting is one of the key activities of an opencast mining. A good blasting results in desired rock fragmentation, less noise and vibration, negligible fly rocks generation which ultimately improves the productivity of loading and hauling equipment and safety. Better blasting is resulted by proper utilization of explosive energy inside the blast hole. Over the years, many attempts has been made by different engineers globally to optimise explosive energy used for the given unit volume of rock. Powder factor (P. F. - tonnes of rock blasted /Kg of explosives used) used to be the measuring tools to evaluate explosives performance. This is basically the lagging indicator and attempts have been made to find out the leading indicator. “Energy Factor Concept” which is a step towards finding a “leading indicator” still today confined to theory and academics. Blasting performance has been a very much critical term for the economics of a mine but unfortunately does not have a yardstick for its measurements. Powder factor has always being used as a post-operative performance indicator of blast efficiency. As per the various literatures, only 18-20% of the explosives energy is being used for actual rock breakage of the 100% available explosives energy. Throughout the world,

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efforts have been made to increase the part of explosive energy which is being used in rock breakage.

There are various techniques available globally like stem plug, ball plug, air bag, para plug etc. to increase utilisation of explosive energy for rock breakage, but these are costly and not being manufactured in India. However, plastic funnel is available in local market and having low cost. So, usage of reverse funnel (Figure 1) is identified as one of the techniques to utilize the explosives energy for more breaking of the rock.

![Figure 1: Plastic funnel](image)

D. Karakus et al. has studied the impact of stemming plug to maximize the utilization of explosive energy to improve blasting efficiency and he has found that there was an increase in explosive energy transmitted to rock mass, resulting in better fragmentation. Rampura Agucha opencast mine of HZL, India and Aditya Limestone mine of Aditya Birla Group, India are also practicing the usage of plastic funnel in blast hole (between explosive and stemming) to improve their blasting efficiency.

**THEORY OF PLASTIC FUNNEL**

Two main purpose of using ‘reverse plastic funnel’ in blast hole between explosive and stemming are as follows:

- Eliminates contamination of explosive from drill cuttings, thus increases the Velocity of Detonation (VOD) at upper portion (near to stemming) of the explosive in blast hole.

Its conic shape creates a ‘Wedge effect’ (Figure 2) guiding more of the blasting energy into the rock rather than upward out of the blast hole. The result is lower explosives consumption, improved rock fragmentation, decreased fly-rock and reduced noise and vibration levels and savings in drilling costs. Hence, to realize the benefit of plastic funnel, a pilot trial was taken at Q SEB, West Bokaro of Tata Steel.

![Figure 2: Wedge effect of plastic funnel](image)
West Bokaro is a Division of Tata Steel Limited. It is a producer of coking coal. The mine came into existence as M/s Bokaro and Ramgarh Ltd in 1947. Coal was initially produced from a small underground mine, which was opened in 1948. In 1972 West Bokaro became a Division of the company. West Bokaro produces raw coals from two opencast mines Quarry-AB and Quarry South Eastern Block (SEB). These are beneficiated in two washeries to produce clean coals which is transported to steel plant at Jamshedpur.

West Bokaro is located about 200 km NW of Jamshedpur in Hazaribagh district. It is about 35 km SW of Hazaribagh town and 26 km NE of Ramgarh town. Geographically, it is located west of Lugu Hill. There are several other coal mines, owned by M/s Central Coal Fields Ltd around West Bokaro. The rain-fed Bokaro river flows through the property. Total area of the leasehold is 4300 acres and contains an estimated mine area reserves of about 180 million tons of coal. The loading station at Chainpur is situated 4 km to the South.

The rock encountered here consists of intercalated beds of sand stone, shales and coal seams belonging to Barakar stage of Damuda series of lower Gondowana system, the age of this coalfield are permo-carboniferous. The general dip of the beds is 2-5 degrees towards southeast but it undergoes great variation locally due to faulting, warping thickening and thinning of the strata. There are total ten shale bituminous medium coking coal seams.

PILOT TRIAL AT Q-SEB COAL MINE OF WEST BOKARO, TATA STEEL LTD

The pilot trial was taken in one of Overburden (OB) bench of Q SEB. The details of the blasting parameters are as follows:

- Total number of holes – 89 Nos (22 holes with funnel)
- Average hole depth -10.8 m
- Hole diameter – 165 mm,
- Funnel diameter (larger diameter of funnel) – 155 mm
- Average burden – 5 m
- Average spacing – 6 m
- Average stemming length – 4 m
- Explosive used – Flexigel

The funnel of size 155 m (dia. of larger circle of funnel) is selected so that it can be easily taken down into the blast hole. Funnel specification is shown in Figure 3.

![Figure 3: Specification of plastic funnel](image-url)
After charging the blast holes with emulsion explosive, two funnels jointed together were inserted (shown in Figure 4) in the blast holes (larger diameter of funnel touches the explosive). The reason behind putting two funnels is to provide strength of the funnel to withstand the explosion pressure. After placement of funnel, stemming is carried out with drill cuttings to pack the remaining length of blast hole. Reverse plastic funnels are placed in 22 blast holes and remaining 67 blast holes are without funnel (shown in Figure 5).

![Figure 4: Inserting reverse plastic funnel in blast hole](image)

![Figure 5: Blasting face at Q SEB, West Bokaro](image)

**Blasting Results**

A. Velocity of Detonation (VOD)

In-hole VOD of the explosive of two holes (one hole with funnel and one hole without funnel) were measured by using Micro Trap VOD Recorder (Figure 6).

![Figure 6: MicroTrap VOD recorder](image)

The MicroTrap VOD Recorder is a portable, high resolution and explosives continuous VOD recorder. The VOD data were captured in the field by Microtrap instrument and the then it is
connected to computer to download the data by using software to generated the VOD of the explosive. The VOD results are shown in Figure 7.

![Figure 7: In-hole VOD results](image)

It was observed that the in-hole explosive VOD is slightly greater in blast hole with funnel compare to hole without funnel which means more strength to explosive

B. Fly Rocks generation

It was observed that the fly rocks were negligible (Figure 8) from holes in which funnels are placed.

![Figure 8: Fly rock generation](image)

**TRIAL AT Q-AB COAL MINE OF WEST BOKARO, TATA STEEL LTD**

Initial pilot trial with plastic funnel in blast hole has shown the promising results in terms of more VOD and lesser fly rock generation. Hence another trial is carried out in OB bench at Q SEB, West Bokaro. The details of the blasting parameters are as follows:

- Total Number of holes – 64 Nos
- Average Hole Depth – 14 m
- Hole Diameter – 165 mm,
- Funnel Diameter (larger diameter of funnel) – 155 mm
- Average Burden – 5 m
- Average Spacing – 6 m
- Average Stemming length – 4.2 m
- Number of row - 3
- Explosive used – Bulk Explosive
Funnels are placed in all blast holes. Blast face is shown in Figure 9. It was observed that, fly rocks generation was very less (shown in Figure 10).

Figure 9: Blasting face

Figure 10: Blast results

CONCLUSIONS AND WAY FORWARD

Utilisation of explosive energy maximization is essential to get good blasting results and to avoid unwanted things like vibration, noise, fly rocks and air over pressure generation. Usage of reverse plastic funnel in blast hole is identified as one of the techniques to maximize the explosive energy utilisation. The two main benefit of using funnels are:

- To eliminate the explosive contamination
- To create wedge effect and hence guiding more explosive energy toward the rock rather than upward.
- Trials with plastic funnel in blast hole have shown the promising results in terms of more VOD and lesser fly rock generation. More trials are planned in overburden benches at West Bokaro to establish the benefits.
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REFERENCES

Glenn Tobin, 2013. The Importance of Energy Containment to the Blast Outcome and Justification for use of Stemming Plugs in Ove Burden Removal (Oresome Products Pty Ltd: Australia)
Microtrap operational Manual, Available from:
PALAEODEPOSITONAL CONDITIONS AND HYDROCARBON SOURCE CHARACTERISTICS OF LIGNITES FROM BIKANER-NAGAUR BASIN (RAJASTHAN) WESTERN INDIA BASED ON ORGANIC PETROGRAPHIC STUDIES

Umar Farooq1, Rimpie Chetia2, Runcie Mathews2, Shalivahan Srivastav1, Bhagwan Singh2, Vikram Singh2

ABSTRACT: In Indian subcontinent Cenozoic lignites are found at several places along the boundary of Indian plate which was the shoreline of palaeotethys. However, the western part of the Indian subcontinent is particularly rich in lignites and shales along with hydrocarbon reserves. Thus, this location of the country is an important producer of hydrocarbon and lies at a significant position in the mineral map of the country. In the north western state of Rajasthan, lignites are reported in association with the Cenozoic sedimentary rocks extending an area of 70,000 sq.kms in Bikaner, Barmer, Nagaur, Jalore and Jaisalmer districts. The present study focuses on the lignite bearing sequence exposed in the Matasukh and Barsingsar lignite mines of Bikaner-Nagaur Basin, Rajasthan. The lignite belongs to the Palana Formation of early Palaeocene age (~66-56 Ma). The lignite seam extends over an area of 2.50 sq.kms and is estimated to have consisted of 10.10 million tonnes of lignites. The lignites of Matasukh and Barsingar lignite mines are studied based on organic petrographic data to elucidate the palaeodepositional conditions and the hydrocarbon source potential. The lignites of Matasukh are predominantly composed of huminite macerals (av. 60 vol. %), followed by moderate liptinite content (av. 23 vol. %). Inertinite macerals (av. 9 vol. %) and mineral matter (av. 8 vol. %) are in lesser proportions. However, in Barsingsar, the lignites are predominantly composed of huminite macerals (av. 74 vol. %), followed by Inertinite macerals (av. 10 vol. %), liptinite content (av. 9 vol.%) and mineral matter (av. 6 vol. %). The dominance of detrohuminite (attrinite + densinite) concerning telohuminite (textinite + ulminite) suggests that the organic matter has undergone a higher degree of degradation; as is also indicated by the frequent occurrence of funginite. In Matasukh, Low TPI and GI values indicate limno-telmatic and mesotrophic-rheotrophic conditions of the palaeomire during the deposition of the lignite forming peat. The deposition took place in varying depositional settings. However, in Barsingsar, the lignites are formed in limnic and rheotrophic conditions. The deposition took place possibly in a marshy depositional setting. Matasukh lignite has huminite reflectance values (av. VRo= 0.26%) indicating that the studied lignites attained ‘brown coal’ as German Standard or ‘lignitic’ stage/rank (ASTM) and is of low rank B (ISO: 7404-5, 2009). Altogether, the lignites of Bikaner-Nagaur Basin show varying petrographic characteristics indicating the variation in the source floral composition, microbial degradation and depositional conditions in different parts of the basin.

INTRODUCTION

Lignite is an important fossil fuel consisting of organic matter which reflects the change in paleoclimatic and paleoenvironmental conditions over geological time (Mitroviae et al., 2016). Microscopic constituents in lignite/coal like maceral and microlithotype were used in various
studies to estimate depositional conditions and peatification of land plant material (Diessel, 1986; Calder et al., 1991; Kalkreuth et al., 1991; Bechtel et al., 2003; Petersen and Ratanasthien, 2011; Bechtel et al., 2014; Stock et al., 2016). Progressive coalification causes not only the rank changes but also chemical composition and properties of the macerals also changes. Oil and gas reserve of the basin is related to source rock, tectonic structure, depositional condition and thermal maturity of the organic matter. Because macerals are the building blocks of coal, it essential to have a proper understanding of their chemical composition, structure, and their decompositional products (Sun et al., 1998; Wilkins and George, 2002). Low-rank coals are dominantly composed of huminite which is isotropic in nature, and anisotropy increases with increase in coal rank. In low-rank coals, huminite significantly influences the technological properties, and this is related to the degree of humification and gelification (Sy´korova´ et al., 2005).

In the Indian subcontinent, the north-western region of Rajasthan is already considered as a highly potential for natural resources especially fossil fuel. The sediments of this region are enriched in carbonaceous sediments such as lignites and shale along with the hydrocarbon reserves. For this reason, this part of the country is an important producer of oil and gas. Lignites from Jalore, Bikaner, Barmer and Jaisalmer districts of Rajasthan are reported from Cenozoic sediments extending an area of 17,000 km². The Matasukh lignite is being commercially exploited by the Rajasthan State Mines and Minerals Limited (RSMML) since 2003 and Barsingsar Lignite exploited by Neyveli Lignite Corporation Limited (NLC). The present study deals with the lignite bearing sequence exposed in Barsingsar mine located in the Bikaner district of Rajasthan and Matasukh mine located in Nagaur district of central Rajasthan. The most used method for evaluation and evolution of coal/lignite deposit is petrographic analysis (Diessel, 1983; Diessel, 1986; Calder et al., 1991; Taylor et al., 1998; Suárez-Ruiz et al., 2012; Stock et al., 2016). Multiproxy analysis of organic matter in lignite has occurred in the last few years which shows most significance in term of economic evaluation and palaeoenvironmental reconstruction (e.g. Bechtel et al., 2005, 2007; Zdravkov et al., 2011; Singh et al., 2013; Stefanova et al., 2013; Životić et al., 2013). The aim of the present study focused on the depositional conditions and hydrocarbon source characterization during peatification of lignite by organic petrography.

GEOLOGICAL SETTING

Barsingsar and Matasukh lignite belong to the Palana Formation of Bikaner-Nagaur Basin. The Barsingsar block is located about 25Kms in a south-west direction from Bikaner city in the State of Rajasthan and falls on the toposheet No.45E/1 of Survey of India having latitude 27°48′59″-27°51′02″N and longitude 73°11′20″-73°11′58″E. The Matasukh lignite mine is located at a distance of about 42 km from Nagaur town between latitude 27°00′N and longitude 74°00′E belongs to the Nagaur-Merta sub-basin of central Rajasthan. This sub-basin is the southern part of the Bikaner-Nagaur Basin with Palana-Kolayat being the northern sub-basin. Bikaner-Nagaur Basin of Rajasthan is elongated in shape and falls in Bikaner and Nagaur districts. The two districts are separated by ‘Arenaceous High’ belonging to Marwar Supergroup. The basin extends for nearly 200 km in E-W direction while its maximum width is 50 km. The northern and southern boundaries the basin are marked by E-W trending faults with basement hight at Dulmera, Suratgarh and the surrounding region. The rocks of Neoproterozoic Nagaur Group (Marwar Supergroup) are overlain by the Cenozoic sediments of continental and marine origin in this basin. These Cenozoic sediments occur, in conformable contacts as Palana, Marh and Jogira formations in ascending order.

The Palana Formation is the sole repository of lignite in this basin and is comprised of grey to dark grey, greenish grey and variegated clays intercalated with fine to medium grained sandstone. Marh Formation is chiefly composed of sandstone having ferruginous stains and
alternate clay beds. La Touche (1902) was first to map the geology of the Palana area and mention the occurrence of lignite (1.2–2.0 m thick) encountered in a dug well. Subsequently, the geological study was carried out by a number of workers and mineral potential was estimated (Heron, 1935; Sethi, 1951; Dutta, 1971). A comprehensive account of the geology of north-western Rajasthan has been provided by Pareek (1984). Based on the study made on the Cenozoic sediments of the Bikaner-Nagaur basin, the geological map has been revised by Dasgupta et al. (1988). Geological Survey of India (GSI) has carried out detailed exploration of the lignite bearing sequence in the basin (Jodha, 2009). The lignite seams with variable thickness have been reported in different boreholes drilled by GSI, Department of Mines and Geology, Rajasthan (DMGR) and Mineral Exploration Corporation Ltd. (MECL). The lignite seams of 0.5–12 m thickness have been intersected in Raneri Block at a depth of 50–150 m (Jodha, 2009). In few boreholes, lignite horizons have also been intersected at a shallower depth. For a better understanding of basin configuration gravity and magnetic surveys were carried out by GSI which has revealed the presence of gravity low having thick lignite bearing Cenozoic sediments. Besides, few more gravity highs and lows have also been identified in the basin by their geophysical survey. The details of the geological succession of Bikaner-Nagaur Basin along with lithology are furnished in Table 1 and the geological map is shown in Figure 1.

Table 1: General stratigraphic succession in the Bikaner-Nagaur basin, Rajasthan (Modified after Ghose 1983)

<table>
<thead>
<tr>
<th>Age</th>
<th>Formation</th>
<th>Lithounits</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pleistocene to Recent</td>
<td>Kolayat Formation</td>
<td>Sand and sandy alluvium ironstone nodule, Sandy calcareous grit kankar, gypsite, ferruginous band, semi-consolidated conglomerate Erratic boulder of quartzite</td>
<td>5-11 m, 1-2 m</td>
</tr>
<tr>
<td></td>
<td>Unconformity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Early to Middle Eocene</td>
<td>Jogaria Formation (Calcareous facies)</td>
<td>Shaly and Marly limestone with foraminifers (Alveolina, Discocyclina Nummulities), Unfossiliferous, white clayey marl</td>
<td>5-10 m, 1 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dirty brown impure limestone with broken shells of Ostrea and foraminifers (Assilina).</td>
<td>1.5 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fuller's earth with shale partings having casts of lamellibranchs and gastropods.</td>
<td>14 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cream and yellowish white limestone full of smaller foraminifers (Nummulites and Assilina) with a thin band of fuller's earth (1-2 m) near base.</td>
<td>75 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yellow shales ochers, marl, etc., with smaller foraminifers (Nummulites, Assilina)</td>
<td>20 m</td>
</tr>
<tr>
<td></td>
<td>Angular unconformity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Palaeocene to Lower Eocene</td>
<td>Marh Formation (Arenaceous facies)</td>
<td>Upper clay horizon with one clay bed.</td>
<td>3-10 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ferruginous sandstone, gritty sandstone and sugary sandstone with white glass sand (local).</td>
<td>60 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Middle clay horizon with five clay beds and sandstone partings.</td>
<td>50 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ferruginous sandstone, gritty sandstone, grit, siltstone.</td>
<td>70 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lower clay horizon with one clay bed. Ferruginous sandstone, gritty sandstone.</td>
<td>1-3 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>various siltstone with leaf impressions (base not exposed).</td>
<td>20 m</td>
</tr>
<tr>
<td></td>
<td>(?) Gradational contact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Palaeocene</td>
<td>Palana Formation (Carbonaceous facies)</td>
<td>Fine grained sandstone, carbonaceous shale and lignite.</td>
<td>120 m</td>
</tr>
<tr>
<td></td>
<td>Base not encountered</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
SAMPLING AND METHODOLOGY

Twenty one lignite samples were analyzed in this study. Samples were taken from the open-pit mine Barsingsar, operated by NLC and open-pit mine Matasukh, operated by RSMML. Among 21 samples, 11 lignites were from Barsingsar, and ten lignite samples were from Matasukh. Four samples of lignite were collected from the top seam-1 (ca. 8 m) and seven lignite samples from the bottom seam-2 (ca. 15 m, Figure 2) of Barsingsar mines. The upper seam is overlain by clay of variable thickness. A shale layer of ca. 3 m separates both lignite seams. Out of the ten samples from Matasukh, four lignite samples were collected from the top seam-1 (ca. 2 m) and six from the bottom seam-2 (ca. 2.75 m, Figure 2). Grey shales of variable thickness underlay both the lignite seams in this area and a sand layer of ca. 0.75 m lies in between both lignite seams.
Petrography

Specifications of ISO 7404-2 (2009) for the micropetrographical analysis were followed. Standards of ISO 7404-3 (2009) and ISO 7404-5 (2009) were followed to carry out maceral analyses and reflectance measurements. Study of macerals was done by using a Leica DM 4500P microscope, at the same time under the normal incident and fluorescence (blue light) under an oil immersion objective (50×) to distinguish the liptinite maceral. The descriptions and nomenclature as provided by Stach et al., (1982) and Taylor et al. (1998) and Šýkorová et al. (2005), are used in this study. With the help of software Petroglite 2.35., counts of 500 macerals were done for the maceral analysis. Measurements of reflectance (100 readings/sample) were made on maceral huminite with the help of Sapphire(0.594% Ro) standard, immersion oil method (Rf:1.518), photometry system (PMT III) and the software MSP 200.

RESULTS AND DISCUSSION

The Palaeocene lignites of the Bikaner-Nagaur Basin have a thermal maturity indicated by Vitrinite Reflectance (VRo) between 0.26 and 0.34% which put them as 'low rank C' coals as per ISO-11760 (2005). The megascopic seam profiles reveal that these lignites have mostly stratified bands, except a few non-stratified bands, of brown to black colour.

Petrographic characteristics

The lignites of Bikaner-Nagaur Basin are dominantly composed of huminite group macerals with subordinate amounts of the liptinite and inertinite groups. Ulminite is sometimes characterised by prominent fractures which are filled up with argillaceous mineral matter or remain empty. Textinite occurs in very low concentrations, especially in Matasukh lignite. Among liptinite macerals mainly liptodetrinite and resinite are recorded though in a few regions show in abundance. The cutinite is represented chiefly by small cutinites. The abundance of sporinite having well developed cell fillings is also observed. Oval to rounded resin bodies sometimes irregular in shape are also noticed which vary in size from less than ten μm to over 100 μm. Few patches of fusinite, with well-preserved cell structure, have also been observed especially in Barsingsar lignites. The cell lumens are sometimes filled with mineral matter and on a few occasions by frambooidal pyrites. At times, the cells are elongated and even oriented. Inertodetrinite is common in both the lignite samples. Argillaceous minerals occupy the cell lumens and fractures. Frambooidal pyrite occurs as single grain as well as clusters. The carbonates are mainly represented by siderite. Characteristic macerals are shown in the photomicrographs (Figure 3).

![Figure 3: Representative photomicrograph of macerals and associated mineral matter in the studied lignite: a) textinite b) corpohuminite c) densinite d) perhydrous huminite e) cutinite f) resinite g) spores (cluster of) h) funginite i) pyrite.](image)
Barsingsar lignite seam
Huminite is the most abundant maceral group (60–86%; av. 76 vol. %) and consists mainly of detrohuminite followed by telohuminite. Detrohuminite is represented by densinite (35–81%; av. 68 vol. %) and attrinite (1–8%; av. 4 vol. %) while telohuminite is mainly contributed by ulminite (1–18%; av. 4 vol. %). Textinite was almost absent. Gelohuminite mainly composed of corpohuminite (1–2%; av. 0.3 vol. %) and gelinite was absent. Inertinite has a relatively low percentage (3–23%; av. 9 vol. %) and liptinite shows low content (3–18%; av. 10 vol. %) in these lignites. The mineral matter varies from 2–18% (av. 6 vol. %). The maceral composition is given in Table 2 and the overall composition of Barsingsar lignite is given in the Figure 4.

Matasukh lignite seam
Huminite is the most abundant maceral group (60–68%; av. 63 vol. %) and consists mainly of detrohuminite followed by telohuminite. Detrohuminite is represented by densinite (15–40%; av. 27 vol. %) and attrinite (10–23%; av. 18 vol. %) while telohuminite is mainly contributed by ulminite (6–26%; av. 10 vol. %). Textinite occurs in very low concentration (0–6%; avg. 3 vol. %). Gelohuminite mainly composed of corpohuminite (3–11%; av. 5 vol. %) and gelinite was absent. Inertinite has a relatively low percentage (5–15%; av. 10 vol. %) and liptinite shows low content (15–28%; av. 21 vol. %) in these lignites. The mineral matter varies from 4–10% (av. 6.00 vol. %). The maceral composition is given in Table 3 and the overall composition of Barsingsar lignite is given in the Figure 5.
Depositional conditions

On the basis of maceral grouping mentioned above, diagnostic macerals as palaeoenvironmental indicators are interpreted. The abundance of huminite group macerals (Figure 6) in the studied lignites suggests that the lignites originated in a wet forest swamp environment (Teichmüller and Teichmüller 1982; Bustin et al. 1983), mainly from arborescent vegetation (Rimmer and Davis 1988). The high amount of huminite macerals with a general predominance of dretrohuminite also indicates that alteration of these peats was mainly controlled by suboxic to anoxic condition and deposition in the peat-forming mires, whilst the relatively low content of inertinite indicates the occurrence of low levels of peat (forest) fire and/or oxidation and the coal having been deposited in waterlogged conditions (Stach et al. 1982; Diessel 1992; Flores 2002; Sykorova et al. 2005; Scott and Glasspool 2007; Petersen et al. 2009; Erik 2011). In the present case, large amounts of dretrohuminite in the studied lignites are considered to be related to both the dominance of herbaceous plants in the palaeomires and the poor preservation of woody substance due to prolonged humification in slowly subsiding palaeomires (Diessel 1992; Petersen et al. 2009; Súarez-Ruiz et al. 2012). In general, the herbaceous plants are consistent to those reported by Ahmed (2004) who described high contributions of dicots and herbaceous monocots and a complete absence of gymnospermic pollen. The presence of large amounts of liptinite group macerals (i.e., liptodetrinite and resinite) suggests an accumulation within forested wet-raised bogs (Ratanasthien et al. 1999; Erik 2011).
Figure 6: Ternary diagram of maceral group composition (huminite-liptinite-inertinite) for analysed Barsingsar and Matasukh lignites.

Petrographic facies could also reflect, to some extent, differences in the type of peat-forming plant communities. On the basis of Tissue Preservation Index (TPI) and Gelification Index (GI), the ratio can be used to determine particular peat-forming environment conditions (e.g. Diessel 1986, 1992; Calder et al. 1991; Kalkreuth et al. 1991; Siavalaset et al. 2009; Jasper et al. 2010; Koukouzas et al. 2010; Životić et al. 2013 and many others). The GI-TPI diagram was firstly proposed by Diessel (1986) for high-rank Australian Permian coals. The type of vegetation and degree of humification may be known through TPI regarding less humified structured and strongly humified unstructured tissue derived macerals (Diessel, 1992). Accordingly, a high TPI indicates high subsidence rate of the basin and predominance of wood derived tissues whereas a low TPI is an expression of the lowrate of subsidence and enhanced humification because of the predominance of herbaceous vegetation in the mire. GI is the manifestation of the degree of gelification of huminite macerals. Gelification requires a continuous presence of water. Fluctuation of water table influences GI due to the formation of inertinites during the drier periods. The Ground Water Index (GWI) indicates the level of ground water (and relative rainfall) during the peat accumulation. According to Amijaya and Littke (2005), there are mainly three types of hydrological condition responsible for formation of mires such as ombrotrophic, mesotrophic and rheotrophic. If the value of GWI < 0.5, 0.5-1 and >1, indicates an ombrotrophic, mesotrophic and rheotrophic hydrological condition, respectively. Type of vegetation present in the mire indicates vegetation index (VI) which depends on the type of peat-forming plant communities (e.g., bushes and trees). To calculate GI, TPI, GWI and VI, it was decided to follow the formula given by Kalaitzidis et al. (2000) which is shown below:

\[
\begin{align*}
GI &= \frac{\text{ulminite} + \text{gelohuminite} + \text{densinite}}{\text{textinite} + \text{attrinite} + \text{inertinite}} \\
TPI &= \frac{\text{telohuminite} + 2\text{corpohuminite} + \text{fusinite} + \text{semifusinite}}{\text{attrinite} + \text{densinite} + \text{gelinite} + \text{inertodetrinite}} \\
GWI &= \frac{\text{corpohuminite} + \text{gelinite} + \text{densinite} + \text{mineral matter}}{\text{textinite} + \text{ulminite} + \text{attrinite}} \\
VI &= \frac{\text{telohuminite} + \text{resinite} + \text{suberinite} + \text{fusinite} + \text{semifusinite}}{\text{detrohuminite} + \text{inertodetrinite} + \text{cutinite} + \text{sporinite} + \text{alginite} + \text{bituminite} + \text{liptodetrinite}}
\end{align*}
\]

The GI vs. TPI plot and the GWI vs. VI plot for the studied samples are provided in the Figure 7a and b. A high GI (2.34-14.4, avg.6.65) and low TPI (0.06-0.62, avg. 0.16) values in the lignite
of Barsingsar mine are indicative of continuous wet conditions in the basin with a slow rate of subsidence during the decay of organic matter. Lignite of Matasukh showing moderate GI (0.90.-2.30, avg. 1.50) and low TPI (0.27-0.77, avg. 0.40) values are indicative of a moderate water column level in the basin with a slow rate of subsidence during the decay of organic matter and represent predominantly topogenous mire conditions. This is well corroborated with higher proportions of huminite over inertinite macerals in the lignites of Barsingsar and Matasukh mine. An estimate of GWI (2.28-29.71, avg. 14.11) and VI (0.07-0.49, avg. 0.17) values in the lignite of Barsingsar mine and estimate of GWI (0.54-2.80) and VI (0.28-0.72) values in the lignite of Matasukh mine are found. Both lignite samples have average GWI value more than one which is indicating the rheotrophic condition. VI value of Matasukh mine is relatively more than the Barsingsar mine which is showing more terrestrial herbaceous plant input in the Matasukh than Barsingsar mine.

Figure 7a: Diagram of GWI and VI indices of the Barsingsar and Matasukh lignites showing palaeoenvironment conditions (Calder et al., 1991).

Figure 7b: GI and TPI diagram indicating the relation of hydrological conditions and the type of mire in Western Rajasthan, India (Diessel 1986, 1992).
HYDROCARBON POTENTIAL

Hydrocarbon generation from source rock is associated with the generation of lighter molecules from the heavy molecules in the original organic matter. In general, the potential to generate hydrocarbon is found to increase from Carboniferous to Cenozoic coals which is related to the evolution of more complex plant communities. However, the depositional condition also plays a major role (Petersen, 2005). The chemical composition of source rock is very important in determining the hydrocarbon potential. The kinds of source vegetation determine this chemical composition. In the present study, the maceral composition of Bikaner-Nagaur Basin lignites is compared with the general chemical composition of macerals which is already known (Mastalerz et al., 2013). One of the limiting factors for hydrocarbon generation potential of any organic matter is the amount of hydrogen-rich macerals incorporated in it. Aliphatic chains in the organic matrix lead to the formation of hydrocarbons. The organic matter derived from marine and lacustrine source rocks (type I and II respectively) are geochemically uniform and contains an abundance of long-chain \( n \)-alkanes, a prerequisite for oil formation. Coaly organic matter (mainly type III kerogen), is composed of heterogeneous and complex higher land plant materials which are richer in oxygen and have fewer long chain \( n \)-alkanes leading to its lower oil generation potential (Petersen, 2005). However, during deposition, the paralic coals are dominant in hydrogen which is affected by the seawater (Petersen and Rosenberg 1998; Sykes 2001), and incorporation of the hydrogen into aliphatic chains may increase the generation capacity.

Aliphaticity of the organic matter apparently shows the richness of hydrogen content in the macerals. Aliphaticity is an important parameter in hydrocarbon potential as higher aliphatic composition of organic material means more the hydrocarbon potential. Qualitative analysis on the functional groups of different liptinite macerals showed defined and prominent aliphatic stretching bands, lower absorbance of aromatic carbon whereas huminite macerals showed prominent aromatic absorption bands (Mastalerz et al., 2013). These are expected differences and result from the aliphatic character of liptinite group macerals in contrast to more aromatic huminite (e.g. Mastalerz and Bustin, 1993). Alginite showed prominent aliphatic stretching bands, their narrow peaks resulting from an overwhelming contribution of \( \text{CH}_2 \) and a very minor \( \text{CH}_3 \) contribution in this region, suggesting long and straight aliphatic chains (Lin and Ritz, 1993). The petrographic studies of the lignites of Bikaner-Nagaur Basin indicate that these lignites are mainly composed of huminite group macerals, particularly detrohuminite. Huminite group macerals are of higher aromaticity within which ulminite and corpohuminite have the lowest and highest aromaticity, respectively (Mastalerz et al., 2013). Liptinite macerals subdominate in the studied lignites. The higher aliphaticity of liptinite macerals and its abundance point to higher oil potential. However, relatively less abundance of liptinite macerals and higher abundance of huminite macerals in Bikaner-Nagaur lignites point to its gaseous potential. Along with the maceral composition, maturity is another important factor determining the ability of hydrocarbon source rock potential. The huminite reflectance (VRo) study of Matasukh lignite ranges from 0.26 to 0.34 suggesting an immature nature (Singh et al., 2017).
### Table 2: Maceral contents (vol. %), mineral matter content (vol. %) and petrographic indices (GI, TPI, GWI, VI) of the studied Barsingsar lignite.

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<th>BR-7</th>
<th>BR-10</th>
<th>BR-12</th>
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Table 3: Maceral contents (vol. %), mineral matter content (vol. %), huminite reflectance (VR₀ %) and petrographic indices (GI, TPI, GWI, VI) of the studied Matasukh lignite

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University of Wollongong, February 2019
CONCLUSIONS

The following observations have been made from the study on lignites from Matasukh and Barsingsar lignites of the Bikaner-Nagaur Basin: The Bikaner-Nagaur lignites are dominantly composed of huminite macerals with the sub dominance of liptinites suggesting prolific inputs from the woody forest vegetation. The high amount of detrohuminite suggests inputs form herbaceous vegetation and the highly active microbial degradation of the peat biomass. Based on the maceral composition and faceis diagrams, it is evident that the lignites of the Bikaner-Nagaur Basin are formed generally in mesotrophic to rheotrophic and limnic to limnotelmatic conditions. However, the differences in the maceral composition from both lignites indicate that there were variations in the source vegetation composition, degree of humification, exposure of the peat biomass, microbial activity and the hydrological conditions experienced in both the mires in space and time. The occurrence of ornamented sporinite and pyrite suggesting near shore conditions of formation. The huminite reflectance indicates the immaturity of the organic matter. However, these paralic coals are potential source rocks for gaseous hydrocarbons on maturity.

ACKNOWLEDGEMENTS

Authors are thankful to IIT(ISM), Dhanbad and the Director (BSIP Lucknow) for providing necessary infrastructure and support for the study. The authorities and staff of RSMML Matasukh, Rajasthan and NLC, Barsingsar are acknowledged for their assistance and cooperation in the field.

REFERENCES


Kalaitzidis, S, Bouzinos, A, Christanis, K, 2000. Paleoenvironment of lignite formation prior to and after the deposition of the “characteristic sand”. In: The Lignite Deposit of Ptolemais. vol. 115. pp. 29–42 (Miner Wealth: Athens)


NUMERICAL INVESTIGATIONS OF INFILLED JOINT SHEAR UNDER DIFFERENT LATERAL BOUNDARY CONFINEMENT CONDITIONS

Libin Gong¹, Jan Nemcik² and Ting Ren³

ABSTRACT: Direct shear is one of the most common laboratory testing methods to study the shear behaviour of infilled rock joints. One of the major concerns is the boundary conditions of infill material in the shear direction, or in other words the “lateral confinement” conditions of the infill layer. In this paper, the effects of various infill lateral confinement conditions on the joint shear behaviour were investigated in the software FLAC. It was found that the interface slip patterns, shear band distribution, and the variations of shear stress and lateral confining stresses, are all dependent on the selected lateral conditions. To ensure a stable model and prevent infill squeezing, two elastic blocks can be glued on the lateral edges of the infill layer. Otherwise, the lateral stress confinement (i.e. constant stress, stress equal to average infill horizontal (XX) stress, and the constant lateral stiffness conditions) are suitable for realistic modelling. Furthermore, the Constant Lateral Stiffness (CLS) conditions lead to asymmetrical confining stresses during shear, and an increase in the stiffness raises the infilled-joint shear strength exponentially.

INTRODUCTION

Shear behaviour of infilled rock joints largely controls the overall stability of a rock mass (Barton et al., 1974). Accurately estimating the joint shear strength is essential in jointed rock engineering practice. So far, related laboratory investigations have been carried out for over 40 years. Most of these studies used direct shear testing which was also adopted in this research. One important concern when conducting the direct shear of infilled joints in the laboratory is the boundary conditions of infill material in the shear direction, referred to here as the “lateral confinement” conditions of the infill layer. Typically, the lateral boundaries are simply set as free or unconfined in the shear direction during testing (Ladanyi and Archambault, 1977; Kutter and Rautenberg, 1979; Papaliangas et al., 1990; Indraratna et al., 1999; Jahanian and Sadaghiani, 2015; Lu et al., 2017), and the infill material could be squeezed out especially when the joint surface is smooth and/or the infill is highly plastic. However, such phenomenon is not likely to occur in the field (Kutter and Rautenberg, 1979; Barla et al., 1987; Barton, 1974).

It seems the direct shear is not the most appropriate testing method for investigating the infilled-joint shear behaviour. Kutter and Rautenberg (1979) attempted to correct the experimental error resulting from infill squeezing during direct shear, by considering the rate of compaction (squeezing). Xu et al. (1988) designed a rotary shear machine for infilled joints, with the lower part of the rock sample and infill material submerged in water. However, no additional hydraulic pressure was applied to the water and thus the infill material was still nearly unconfined at the lateral boundary. To better simulate the real shear conditions of natural infilled joints in the laboratory, Barla et al. (1987) designed a simple shear testing apparatus rather than using the direct shear method, where the infill squeezing was prevented by a membrane that enabled confining stress to be applied at the boundaries. On the other hand, Hatzor and Levin (1997), Indraratna et al. (2008), and Khosravi et al. (2016) adopted a triaxial apparatus to study the infilled-joint shear behaviour, where the cylindrical sample with a single inclined joint infilled

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with soil was used. The “lateral” boundaries of the infilled soil were thus constrained by the membrane at a certain confining pressure.

Different lateral boundary confinement conditions can be applied to the infill layer when conducting the laboratory shear tests of infilled joints, yet comparisons among various confinement conditions in the literature are rare. The influences of different lateral confinement conditions on the joint shear behaviour in the laboratory and the deviations between the laboratory applied conditions and the real field conditions tend to be ignored. Compared with laboratory methods, the numerical method is much easier and to use and more efficient at conducting preliminary comparison studies on a wide range of different conditions. In this study, the FLAC (Fast Lagrangian Analysis of Continua) software (Itasca, 2011) was adopted to investigate the effects of lateral confinement on the infilled joint shear behaviour on a laboratory scale.

**PROBLEM STATEMENT AND MODELLING PROCEDURES**

Typically there are two stages in conducting laboratory shear tests of infilled joints. The infill material is initially compacted (when unsaturated) or consolidated (when saturated) to reach equilibrium followed by the second stage where shear movement begins. Ideally, the lateral confinement of the infilled joint sample should remain the same during these two stages to ensure stabilisation of the infill layer just before shear begins. Numerical tests of the first stage (before shearing) were trialled under different lateral conditions to select practical ways of preventing the infill squeezing out at the sample boundary. Trials of 9 lateral confinement methods using FLAC are illustrated in Figure 1.

![Figure 1: Schematics of various infill boundary conditions](image-url)
Note: the abbreviations used in Figure 1 are explained here: (a) FB - the Free Boundary condition; (b) FLD - the Fixed Lateral Displacement condition; (c) IGLS - the Interface Glued at Lateral Sides condition; (d) EGB - Elastic Glued Blocks confined condition; (e) UEB - the Unglued Elastic Blocks confined condition; (f) SEG – the Single-side-glued Elastic blocks confined condition; (g) CLS - the Constant Lateral Stress condition; (h) ALS - the Average Lateral Stress confined condition; (i) CCS - the Constant Confining Stiffness condition.

A fine grid of 99 × 112 zones was set up, representing a horizontal infilled joint model of the typical laboratory scale of 100 mm × 110 mm with the 10 mm thick infill layer confined by two 50 mm high rock blocks. A constant vertical stress of 1.5 MPa was applied perpendicular to the infill joint to simulate CNL conditions typically used in laboratory direct shear tests. Initially, the displacements of both the left and right sides of the upper rock block and the left side of the lower block were fixed in the lateral direction while the bottom boundary was fixed in the vertical direction. Note that the orientation of “left” and “right” mentioned here as well as in the following contents are defined in Figure 1. The lateral boundaries of the infill material were confined depending on the investigated situation.

As this study focused on the infill boundary confinement variations, the upper and lower rock parts were defined as an isotropic elastic material. The infill layer was modelled as a Mohr-Coulomb (M-C) material. Deformability and strength properties required in FLAC for both the rock part and the infill part are listed in Table 1. The selected values for the infill material represent a relatively highly plastic soil. Unglued interfaces in FLAC were used to construct the contact planes between the infill material and the rock surface, where the Mohr-Coulomb shear-strength criterion applies.

**Table 1: Infilled joint specimen properties required in FLAC**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Rock</th>
<th>Infill material</th>
<th>Rock-infill interface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constitutive model</td>
<td>Isotropic elastic</td>
<td>M-C model</td>
<td>Coulomb sliding and/or Tensile separation</td>
</tr>
<tr>
<td>Dry density (kg/m³)</td>
<td>2700</td>
<td>2000</td>
<td>-</td>
</tr>
<tr>
<td>Bulk modulus (Pa)</td>
<td>10×10⁹</td>
<td>7.8×10⁹</td>
<td>-</td>
</tr>
<tr>
<td>Shear modulus (Pa)</td>
<td>4×10⁹</td>
<td>5.8×10⁹</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.32</td>
<td>0.2</td>
<td>-</td>
</tr>
<tr>
<td>Cohesion (Pa)</td>
<td>-</td>
<td>14×10⁹</td>
<td>14×10⁹</td>
</tr>
<tr>
<td>Drained friction angle</td>
<td>-</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Dilation angle</td>
<td>-</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Tension limit (Pa)</td>
<td>-</td>
<td>1×10⁹</td>
<td>0</td>
</tr>
<tr>
<td>Normal stiffness (Pa/m)</td>
<td>-</td>
<td>-</td>
<td>1.55×10¹¹</td>
</tr>
<tr>
<td>Shear stiffness (Pa/m)</td>
<td>-</td>
<td>-</td>
<td>2×10⁸</td>
</tr>
</tbody>
</table>

Figure 2 and Figure 3 demonstrate the deformation, vertical stress and failure states after initial equilibrium of the tests at various lateral conditions. Detailed descriptions for each condition are as follows:

- The Free Boundary (FB) condition

The lateral boundary of the infill layer can be unconfined, defined here as the Free Boundary (FB) condition (Figure 1a). However this condition is unrealistic. It is nearly impossible to prepare a sample that can withstand a steady-state loading especially when the joint surface is smooth and/or the infill material is highly plastic. This problem is evident in Figure 2(a) where the infill material was squeezed out significantly after only 300 calculation steps. If stepped further, these deformations would continue because relatively large velocities and maximum unbalanced forces within the infill did not converge towards the static equilibrium. Figure 3(a) indicates that after running for 300 steps two noticeable stress-relaxed areas occurred around the infill lateral boundaries due to the infill yielding and squeezing out.
• The Fixed Lateral Displacement (FLD) condition

It is reasonable that the lateral boundary of the infill is set as displacement-fixed during the initial stage (Figure 1b). This condition is the same as the lateral confinement conditions applied in the typical laboratory experiments, as well as the conditions adopted by Indraratna et al. (1999). Equilibrium and a stable model can be obtained under this Fixed Lateral Displacement (FLD) condition, based on the velocity plots and the maximum unbalanced force history as shown in Figure 2(b). Subsequently, no significant stress concentration, relaxation or infill yielding occurred inside the model, as shown in Figure 3(b). However, the confinement must be removed when shearing starts to allow shear deformation of the infill. Such removal will cause a sudden change of the infill boundary conditions, leading to possible continued deformation of the infill layer, especially when the infill is highly plastic. Hence this condition is not suitable to study various joint scenarios in the field.

• The Interfaces Glued at Lateral Sides (IGLS) condition and the Elastic Glued Blocks (EGB) confined condition

As shown in Figure 1(c), both left and right sides of the infill-rock interfaces can be glued in the FLAC model to prevent possible squeezing of the infill. This situation is defined here as the Interface Glued at Lateral Sides (IGLS) condition. By conducting a trial modelling with such condition as shown in Figure 2(c), it seems impossible to reach equilibrium before shear takes place for joints infilled with relatively high plastic material. Two stress-relaxed areas formed surrounding the infill lateral boundaries as shown in Figure 3(c). They were similar to but relatively smaller than the stress-relaxed areas shown in the Free Boundary (FB) condition.

Nevertheless, initial equilibrium of the infilled joint model can be achieved by setting two elastic glued blocks on the lateral edges of the infill layer as shown in Figure 1(d). This method is referred to as the Elastic Glued Blocks (EGB) confined condition. The side infill confining blocks were modelled as elastic material having the same modulus as the infill layer. Figure 2(d) shows the model after reaching equilibrium under this condition. It can be seen that the squeezing of the infill was avoided. However, significant stress concentrations occurred around the infill lateral boundaries, as shown in Figure 3(d).

• The Unglued Elastic Blocks (UEB) confined condition

Another technique for preventing squeezing of the infill material is a pair of elastic blocks that can be placed at the infill edges without glued interfaces. This method is defined here as the Unglued Elastic Blocks (UEB) confined condition. Figure 1(e) shows the corresponding boundary conditions. The infill deformation was already unacceptable before the static equilibrium was achieved, as shown in Figure 2(e). The Mohr-Coulomb part of the infill started yielding at both left and right ends, and stresses were concentrated around the infill lateral boundaries, as indicated in Figure 3(e).

• The Single-side-glued Elastic Blocks (SEB) confined condition

The elastic blocks can be put on lateral edges with glued interfaces only on one block side. This condition is also a logical trial, as shown in Figure 1(f). However, despite that initial equilibrium could be reached under this condition, relatively large deformation occurred on the unglued edges of the infill (Figure 2f), which is considered unreasonable. As expected, Figure 3(f) demonstrates extreme stress concentrations occurring around the glued interface edges.
Figure 2: Initial models for various infill confinement conditions before shear
Figure 3: Vertical (YY-) stress contours of the initial model at different infill lateral boundary conditions

- The Lateral Stress Confined (LSC) conditions

The lateral boundaries of the infill layer can be confined by either varying lateral stresses or constant lateral stresses, defined here as the Lateral Stress Confined (LSC) conditions. Obviously these conditions are more close to the real practice compared with other conditions mentioned above. Possible Lateral Stress Confined (LSC) conditions on the infill lateral boundary are:

(a) the Constant Lateral Stress (CLS) condition, as shown in Figure 1(g);

(b) the confining stress equal to the average values of the infill stress component in the shear direction during shear, referred to as the Average Lateral Stress (ALS) confined condition, as shown in Figure 1(h). This condition was trialled to enable simulation of field conditions where no boundaries are present.

(c) the Constant lateral Confining Stiffness (CCS) condition, as shown in Figure 1(i).

Note that the conditions in the stage "before shear" of the above three cases are actually the same. The difference emerges when shear starts. Figure 2(g) and Figure 3(g) show the
initial equilibrium models for these confining stress conditions. As expected, no apparent infill squeezing, yielding and stress concentration / relaxation occurred during the equilibrium.

In summary, appropriate lateral conditions in the loaded models before shear may only be the Elastic Glued Blocks (EGB) confined condition, the Constant Lateral Stress (CLS) condition, the Average Lateral Stress (ALS) confined condition, and the Constant Confining Stiffness (CCS) condition, listed above. In the following sections, the infilled-joint shear tests were run based on the initial equilibrium models under these four lateral boundary conditions respectively. A horizontal velocity of $1 \times 10^{-8}$ m per numerical step was applied to the lower block to produce a shear displacement. Planar joints were modelled in all the cases for convenience. Related FLAC codes were attached in Appendix 1, together with extra codes for generating rough surface with various JRC values. Comparisons between the Elastic Glued Blocks (EGB) and the Constant Lateral Stress (CLS) conditions were made. The differences in three distinct Lateral Stress Confined (LSC) conditions were explored, i.e. the Constant Lateral Stress (CLS), the Average Lateral Stress (ALS), and the Constant Confining Stiffness (CCS) conditions. Furthermore, more levels of Constant Confining Stiffness (CCS) were simulated to investigate its influence on the joint shear strength.

**EFFECT OF LATERAL CONFINEMENTS**

**Comparisons between Elastic Glued Blocks and Constant Lateral Stress confined conditions**

The Elastic Glued Blocks (EGB) and the Constant Lateral Stress (CLS) cases are compared here. In the first case, two elastic blocks were placed on both lateral edges, and their contacts with rock were glued. In the second case, a constant confining stress of 1.5 MPa was applied to the infill at the boundaries, which was the same as the initial stresses in all directions within the model.

As shown in Figure 4(a), it is clear that the EGB condition generated a much higher shear stress compared to the CLS condition. A residual state was reached at 6 mm of shear displacement for the CLS condition, while the shear stress grew almost linearly for the EGB condition without failure. This is understandable because under EGB, shear slip along the infill-rock interfaces is restrained due to glued edges, and the elastic blocks continuously provide shear resistance without failure. However, for the CLS condition, the interface is free to slip. Figure 5(a-b) shows the distinct patterns of the interface slips under different conditions. In addition, the shear bands formed inside the infill material are totally different. For the EGB condition, plastic deformation is only distributed within the Mohr Coulomb part of the infill. Plastic yielding is concentrated at the lateral edges close to the elastic blocks as well as at two parallel inclined shear planes inside (Figure 5a). However, for the CLS condition, a shear plane is formed along the diagonal of the infill layer, as shown in (Figure 5b). Also for the EGB condition, both the vertical and horizontal stresses were highly concentrated at the infill lateral edges (Figure 5a) when compared with the stress distributions under the CLS condition (Figure 5b).

**Comparisons of three infill Lateral Stress Confined (LSC) conditions**

Three Lateral Stress Confined (LSC) conditions were compared against each other, including the Constant Lateral Stress (CLS) confined condition, the Average Lateral Stress (ALS) confined condition, and the Constant Confining Stiffness (CCS) condition with CCS = 0.3 MPa/mm.

Figure 4(b) shows that CCS produced a higher shear stress than both CLS and ALS. The CLS and ALS had almost the same shear strength. The distinction is that the shear stress held
steady after peak for the CLS, while the ALS had an obvious strain-softening behaviour. In addition, the distribution of the shear bands within the infill is different, as shown in Figure 5(b-d). Under the CLS and ALS conditions, shear failure occurred mainly inside the infill layer, approximately along its diagonal; while for the CCS condition, shear failure occurred almost along the infill-rock interface on the moving side. Accordingly, shear slip occurred on the edges of both upper and lower interfaces under the CLS and ALS as shown in Figure 5(b-c), while for the CCS condition, interface slip occurred only along the lower joint surface (Figure 5d). In terms of stress distributions, both the vertical and horizontal stresses were much more concentrated at the CCS condition than the CLS and ALS, while the CLS had slightly higher maximum vertical and horizontal stresses than the ALS.

Figure 6(a) shows the variations of applied lateral confining stresses for each condition during shear. A horizontal line was plotted for the Constant Lateral Stress (CLS) condition. The curve for the Average Lateral Stress (ALS) condition shows that average values of the horizontal stress within the infill decreased linearly with a gentle slope before peak shear stress was reached, and then dropped steeply after the peak. On the other hand, the Constant Confining Stiffness (CCS) condition demonstrates different confining stress trends between the left and right sides of the infill. The confining stress increased almost linearly on the right side, while for the left side, the confining stress decreased first, and then remained stable after moving for 5 mm, before a slight increase when shear stress reached the peak value. Also the plotted Constant Confining Stiffness (CCS) curves were much steeper when compared with the Average Lateral Stress (ALS) condition.

**Figure 4:** Shear stress vs horizontal displacement curves. Note: EGB = Elastic Glued Blocks, CLS = Constant Lateral Stress, ALS = Average Lateral Stress, CCS = Constant Confining Stiffness

**Effect of Constant Confining Stiffness conditions on the joint Shear Behaviour**

The joint shear behaviour under different levels of Constant Confining Stiffness (0.3, 3, 30 MPa/mm) was investigated. Figure 5(d-f) shows that in all three stiffness levels, the shear failure mainly started along the lower joint surface, and shear slips occurred only along the lower interface. There seems to be not much difference in stress distributions when the CCS value changes. On the other hand, the shear direction significantly influences the variation trends of the confining stresses, as shown in Figure 6(b). On the right side, the confining stress increases with the increase of CCS, while on the left side, the confining stress decreases when the CCS becomes higher. Basically, the shear stress increased when the CCS increased, as
shown in Figure 4(c). Figure 7 shows that the CCS value influences the shear strength in an exponential way, and an empirical equation was fitted to this relationship:

$$\tau = 0.5264 \cdot (K + 1 \times 10^{-6})^{0.0101}$$  \hspace{1cm} (1)

Where $$\tau$$ is shear strength, and $$K$$ is the value of lateral stiffness (MPa/mm).

**Figure 5**: Stresses, plastic states, and interface slips after shearing at various lateral boundary conditions (black stars and ellipses indicate interface slips)
Figure 5 (continued): Stresses, plastic states, and interface slips after shearing at various lateral boundary conditions (black stars and ellipses indicate interface slips)
SUMMARY

This paper illustrated the differences in shear behaviour of infilled joints with various infill lateral boundary confinement conditions using the numerical modelling software FLAC. These results indicate that the lateral boundary condition of the infill layer significantly influences the joint shear behaviour. When conducting laboratory direct shear tests of infilled joints, it is essential that the boundary influences should be considered carefully, and an appropriate lateral confinement condition similar to the field be applied. Some important conclusions can be drawn:

- To prevent infill squeezing and establish a stable infilled joint model, certain lateral boundary confinement condition should be applied. Practical lateral boundary conditions were proposed when numerically modelling the direct shear tests of infilled joints. For example, two Elastic Glued Blocks (EGB) can be set on the lateral edges of the infill layer, with both sides of the infill-block interfaces glued. Otherwise, either varying or constant lateral stress confined conditions are suitable to achieve the stable conditions for joints infilled with weak materials.

- Modelling results clearly show that the lateral boundary conditions of the infill layer play a significant role during shear. The patterns of interface slip, stress and shear band distribution, and the changes in shear stress and lateral confining stresses, are different under various boundary conditions.
• In terms of the Constant Confining Stiffness (CCS) conditions, it seems that the CCS value has little effects on the modelled stress distribution. In addition, since the joints shear from left to right, the confining stress on the right side increases during shear, while the stress on the left side decreases. Also, with the rise of applied lateral stiffness, the shear strength of the infilled joints increases exponentially.

There are some limitations in this numerical study. The following recommendations are suggested.

• The study focused on the possible lateral confinement conditions applied in the numerical models only, and corresponding laboratory procedures were not studied. For laboratory practice, such conditions can be achieved by using elastic membranes at the lateral boundaries, with a sealed system to apply either constant or varying lateral stresses, for example the shear apparatus adopted by Barla et al. (1987) or the conventional triaxial testing system.

• The shear behaviour of infilled joints was only modelled on a laboratory scale, representing a small segment of large scale joints in the field. The deviations between the laboratory applied boundary conditions and the real field conditions should be well recognised. In addition, boundary effects i.e. the influences of stress concentration / relaxation at the infill lateral boundaries on the laboratory shear testing were not studied. These are largely dependent on the applied boundary confinement conditions and the relative sizes between the infill thickness, joint asperity height and the joint surface dimensions. Also, scale effects were not considered. All these boundary and scale effects need to be fully appreciated, so that the appropriate laboratory boundary conditions and sample sizes can be chosen to accurately represent the field conditions. An empirical method of correcting the laboratory test results can also be implemented. In the future study, these effects under each lateral confinement condition can be investigated numerically, considering that numerical methods are suitable for modelling much larger field-scale cases.

• The laboratory tests were only simulated in two dimensions. The influences of the infill lateral confinements in the way perpendicular to the shear direction were not investigated. Further three-dimensional simulation should be carried out to examine the viability and accuracy of the conventional confinement conditions in the other direction (typically displacement-fixed boundary), when conducting the laboratory direct shear tests of infilled joints.

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REFERENCES


THE IMPACT OF INTRUSION TO COAL CHARACTERISTICS IN TANJUNG ENIM, MUARA ENIM FORMATION, SOUTH SUMATRA

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ABSTRACT: Tanjung Enim is one of the most well-known coalfields in Indonesia. It is situated in the middle of the South Sumatra Basin while the main coal produces from the lower group of Muara Enim Formation, Miocene-Pliocene in age. The lower group of the Muara Enim Formation consists of five main seams overlying the Air Benakat Formation and intercalate with claystone, sandstone and tuffaceous sandstone. Intrusion existence and volcanism influence in the formation differs it from the coalfields in Borneo. Batholith and sill intrusion is example of volcanism activities in the area. This is a product of subduction phenomena which activate the ring of fire and Bukit Barisan in South Sumatera.

Research conducted by geological mapping and analysis of proximate data. Generally, coal lithotype in Tanjung Enim has varied characteristic. Coal near intrusions characterized by black color, bright 80% dull 20%, brittle, black streak, pyrite mineral was filled in cleat, conchoidal cleavage. Analysis indicates that calorific value near intrusion area has higher value than coal without intrusion effect. Calorific value of the coal near intrusion ranged from 7330 kCal/kg to 7855 kCal/kg. On the other side, coal from other location which is 3.5 km far from the intrusion has varied calorific value ranged from 4597 kCal/kg to 5099 kCal/kg of coal as received.

INTRODUCTION

Coal characteristics depend on the environment in which they were a deposited and preserved. The depositional environment leads to maceral type, composition of coal and chemical effect on the coal. One of the most significant associated factors in the coal facies is igneous rock intrusions (Thomas, 2013) as the place where the coal is formed and preserved could affect the coal maturity. Thus, the understanding of depositional environments and geological position of coal deposit is really imperative to characterize the coal and predict the characteristic pattern within geographical areas.

Tanjung Enim was one of the coal-rich areas located in the center of the South Sumatra Basin. Coal resource mainly comes from lower members of Muara thwEnim Formation which was deposited in Miocene-Pliocene. Muara Enim Formation has numerous coal seam but only 5 coal seam that are the main focus, there are the Mangus, Suban, Petai, Kladi and Merapi Seams. Three out of the five seams are regularly the target to mine due to thickness and broadenise. Those three seams consist of Mangus, Suban and Petai Seams. In some areas, Suban and Petai seams are split into two different seams while Mangus always found as two split seams in the whole area of Tanjung Enim. Enim River crosses from north to south and divide Tanjung Enim as 2 parts. Western part is known as Tambang Air Laya and Muara Tiga Besar mine. Otherwise, eastern part named as Banko mine. This paper will focus on the Western side due to its complexity caused by intrusions.

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Generally, the geological conditions in Tanjung Enim were relatively simple. But different cases happen in Tambang Air Laya where intrusions exist and affect the coal post-deposition characteristics. Magma intrusion caused temperature increased to near rock formation and alter the structural properties of the formation that will lead to increasing of pressure. It is known that temperature and pressure are two main factors that affect coal quality. The intrusion has also caused the formation of some new minerals, particularly in the coal which is highly metamorphosed Amijaya, (2006). Investigation of intrusion patterns, altered coals commonly showed increasing ash, vitrinite reflectance and its anisotropy and decreasing volatile matter adjacent to intrusions (Yao, 2011). Therefore, coal quality in Tambang Air Laya has a higher value than coal in other mine units with complex distribution of broad quality range in areas as indicated in Figure 1.

This paper will uncover the coal characteristics especially quality distribution on the effect of intrusion body and intrusion distance to quality in specific drillhole.

![Figure 1: Location of Airlaya Coal Mine marked with Red Mark](image)

**DATA AND METHOD**

This paper uses several methods applied to discover the implications of intrusion on coal characteristics. First, geological mapping conducted to reveal the geological conditions including lithology features that appear in the area, secondly subsurface lithology also acquired by borehole data to have a better description of characteristic of the coal. Third, proximate analysis conducted from each coal sample to recognize the quality of coal such as inherent moisture value, ash, total sulfur and ultimately calorific value. Finally yet importantly, statistical approach used to know the average deviations of calorific value per distance of coal from the intrusion body. Figure 2 shows boreholes locations in the Tambang airlaya.
RESULT AND DISCUSSION

Tanjung Enim coal resources come from the Muara Enim Formation that consists of Sandstone-Claystone intercalated with coal seam. It was deposited during a regression phase in the South Sumatra Basin. Thus, it has a huge deposit of coal resources within the formation due to a deltaic environment and organic abundance. Geological features of the South Sumatra Basin are unique. It has marine to non-marine successions, deep marine to volcanic environments. Intrusions appear as post-depositional features. Volcanic and intrusions in the South Sumatra Basin appeared as part of convergence forces between the Hindia and Eurasia plate. This event leads to the formation of mountain chains (in this area represented by Bukit Barisan).

Research conducted by geological mapping at first, continued with proximate data analysis. From geological mapping, geology units of the field were obtained which consisted of sandstone, claystone, coal and andesitic intrusions. While proximate analysis resulted to coal quality primarily in calorific value. Coal in the southern side of the field has a higher value of calories. It is affected by heat produced from intrusion existence that supports the coal maturation. The closer the coal to the intrusion, the more calorific value it has. The higher quality coal reaches up to 7800 kcal/kg (as received), while lower value of calorie only reach 4597 kcal/kg (as received). This data is shown in Figures 3 to 7.

Intrusion and coal maturity also implied to coal characteristics as description. Premium quality of coal represented by black color, bright 80% dull 20%, brittle, black streak, pyrite mineral filled in cleat, and conchoidal cleavage. Otherwise, coal without intrusion effect has brownish black color, massive, brown streak, friable and relatively clean from mineral.

It is recommended to explore more to the southern area due to higher quality in order to get more premium quality and more profit. For further research, the writer suggests to conduct research about trace element abundance due to intrusion effect.
Figure 3: Change of Seam A1, A2, B1, B2, and C calories number due to increasing of distance to the North from intrusion

Figure 4: Calories number at A1 and B2 Seam in the different distance to the intrusion, and change of A2 and B1 Seam calorie value due to increasing of distance to the East from intrusion
Figure 5: Change of A1, A2, B, and C Seam calories value due to increasing of distance to the West from intrusion

Figure 6: Change of A1, A2, B, and C Seam calories value due to increasing distance to the South from intrusion
Figure 7: Coal Quality distribution of Seam A1, A2, B1, B2, and C model based on calorie number, showing that higher quality coal is located near intrusions. Red colour represents coal with 7200 kcal/kg, Light Brown colour represents 5000 kcal/kg coal.

CONCLUSIONS

Based on field and data observations, it can be concluded that the field has five main seams with slight quality difference in vertical relationship but huge in lateral area. It is affected by the intrusion existening in the southern area of the field. Coal near intrusions has a higher quality rather than coal that is far from intrusion body. These coals also have difference in characteristics. They are brighter and more black yet brittle than coal without intrusion effects.

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REFERENCES


Gafoer, S, Cobrie, T, Purnomo, J, 1986, Geologic Map of Lahat Quadrangle


PALEO-ENVIRONMENT MODEL OF SUB-BITUMINOUS COAL SEAM BASED ON MACERAL DATA ANALYSIS IN BANKO BARAT

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ABSTRACT: Depositional model are a substantial way to to achieve a better understanding about coal properties and seam characteristics. There are many ways to interpret depositional models, one of them is by analysing macerals of the coal obtained from the core. Tissue Preservation Index (TPI) and Gelification Index (GI) plotted in the formula of Diessel (1986) to know the depositional environment. Maceral group observed from coal in Banko Barat show dominant Vitrinite content such as telovitrinite and dextravitrinite. Cutfinite and Resinite appear as a part of Liptinite while Funginit-Semifusinit as a part of Inertinite. These group of maceral was a character of Tertiary Coal Seam. High value of Vitrinite and Lower percentage of liptinite/inertinite indicate that peat swamp in research area was situated in unstable basin between upper delta plain and lower delta plain. Thus, the depositional environment shifted, causing mineral matter filling in the coal.

From the TPI versus GI formula, depositional environment achieved and show that the coal seams were deposited primarily in a Limited Influx Clastic Marsh. At some point, depositional environmental change to Limno Telmatic and Wet Forest Swamp. TPI values from research area varied ranging from 0.022 to 3.36. On the other hand, GI value of the coal ranged from 11.54 to 79.58. These are happened as the effect of high content of Vitrinite Maceral (approximately 89.72%) and Low content of Inertinite (about 3.91%).

INTRODUCTION

Banko Barat is one of the coalmine concessions in Tanjung Enim, South Sumatra (Figure 1). It has five coal seams that produce a coal product. Coal rank in this field is between lignite to sub-bituminous. Calorific value ranged from 4600 kcal/kg to 5500 kcal/kg as received in this field. Coal properties and seam characteristics were mainly affected by maceral and mineral matter filled in the coal. These features belongs to specific depositional environments (Figure 2) and the percentage of each maceral unit (Vitrinite, liptinite, Inertinite) and mineral matter. Thus, it is imperative to get a better insight about coal properties and the seam distribution in the field.

The percentage of maceral of a coal seam has a firm connection to depositional facies when the peat and coal accumulated, so maceral analysis could be used to reconstruct the coal depositional environment.

Coal samples were obtained from the core of a borehole. From this core, some samples were taken to laboratory to conduct maceral analysis. Maceral composition analysis was conducted to obtain the percentage of maceral content of the coal sample. A reflected light microscope was used along with a point Counter. Observation of coal maceral compositions conducted in 1000 points according to ASTM D2799-13. In normal reflected white light mode, maceral type could be determined based on degree of reflectivity. Inertinite was the most reflective type of maceral, while liptinite was on the contrary. Reflected fluorescence mode used to observe maceral compositions in liptinite. Fluorescence intensity depleted as the increasing of organic

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rank/maturity (Cook, 1982; Teichmuller, 1985). It conducted as the adjustment to Diessel Diagram (1986) which will be used to interpret depositional environment in Banko Barat.

Figure 1: Banko Barat Location as one of Bukit Asam Coalmine Concession (number 3)

Figure 2: Coal Depositional Environment Model (Horne, et al., 1978)

DATA AND METHOD

At the final step, Tissue Preservation Index versus Gelification Index was applied to determine the exact depositional environment.

Tissue Preservation Index (TPI) determines the percentage of tree density within the area. The more the value of TPI, the more abundance of tree. On the other side, Gelification Index (GI) were useful to determine whether the environment was dry or wet. It shows how much water was involved in the environment. The more water involved, the more Vitrinite produced in coal compositions. This diagram (Figure3) was composed by Tissue Preservation Index values (x axis) and Gelification Index (y axis). Low value of TPI and GI was interpreted as deposited on Open Marsh while the higher value of both TPI and GI shows Wet Forest Swamp. Low value
of TPI and high value of GI are determined as Limited influx Clastic Marsh. High values of TPI and low values of GI as Dry Forest Swamp.

![Depositional Environment Analysis Diagram Modified from Lamberson (1991)](image)

**RESULT AND DISCUSSION**

Observation on maceral groups showed dominant vitrinite content, where almost all samples dominated by the vitrinite group such as telovitrinite and detrovitrinite as much as 89.72%. On the other side, liptinite group (such as cutinite and resinite) and inertinite group (such as funginite and semifusinite) has lower percentage approximately 3.91%. These groups of maceral are an example of tertiary coal character. High value of Vitrinite and low value of liptinite/inertinite indicated that peat swamp in Banko Barat located at unstable basin between upper delta plain and lower delta plain. Depositional environment shifting had an effect on this unstable basin and caused the formation of mineral matter filling in the coal.

TPI value varied from 0.22 at the lowest (BTR 09 seam B1) and 3.36 at the highest (BTR 15 seam B1). GI value also varied from 11.54 (BTR 08 seam A1) to 79.58 (BTR 07 seam B2).

From TPI vs GI plots, it can be concluded that Seam C was deposited mainly in Limited Clastic Marsh. Along with the time, seam B2 was deposited in Limited Clastic Marsh but with some samples in Limno-Telmatic Marsh. Seam B1 was more Telmatic with the regression period of deposition. Seam A2 was not different from seam B1. Finally, transgression happened again while seam A1 was deposited in Limited Clastic Marsh, back to the condition where seam C deposited.

Figure 4 shows that all coal of seam B1 was rich in Vitrinite content. Vitrinite content has lateral relationship with Gelification Index. The majority of coal B1 has low Tissue Preservation Index just one of the sample has a higher value. In conclusion, seam A1 was deposited mainly on Limited Clastic Marsh and one of sample show Limno Telmatic as depositional environment.
In Figure 5, a diagram of Seam A2 showing that all coal of seam A2 was rich in Vitrinite content. The majority of coal A2 has low Tissue Preservation Index although some of them has higher value. In conclusion, seam A2 was deposited mainly on Limited Clastic Marsh with some sample show Limno Telmatic as depositional environment.

Figure 6 shows that all coal of seam B1 was rich in Vitrinite content. The majority of coal B1 has a low Tissue Preservation Index although some of them have higher value. In conclusion, seam B1 was deposited mainly on Limited Clastic Marsh and Limno Telmatic.

In Figure 7, a diagram showing that all coal of seam B2 was rich in Vitrinite content. The majority of coal B2 has low Tissue Preservation Index although some of them have higher values. In conclusion, seam B2 was deposited mainly on Limited Clastic Marsh and some sample show Limno Telmatic as depositional environment.
Figure 6: TPI vs GI diagram of seam B1

Figure 7: TPI vs GI diagram of seam B2

Figure 8: TPI vs GI diagram of seam C
Diagram of seam C shows that all coal of seam C was rich in Vitrinite content. All of coal B1 has low Tissue Preservation Index. In conclusion, seam C was deposited on Limited Clastic Marsh.

**CONCLUSIONS**

Liptinite and Inertinite were rarely found in samples from the field, while Vitrinite was very abundant. Vitrinite rich coal shows that this coal was deposited in a water-rich environment. This coal is still categorized as lignite type despite has high value of Vitrinite. It is due to low reflectance of the Vitrinite.

Observations and analysis of TPI vs GI in all samples resulted in the interpretation that the main coal seams in the Banko Barat were deposited in Limited Clastic Marsh. It is the environment where tree density is low and water was abundant. In some samples also found characteristics that shows more tissue preserved where indicated it deposited on more tree density.

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