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PREFACE

The coal mining industry in Australia and worldwide in general has progressed dramatically from a labour-intensive industry to highly mechanised, which has led to an increase in production, improved productivity with ever increasing environmental challenges. The Coal Operators Conferences have proven to be a successful platform in introducing many innovative technologies to the industry and still maintains leadership as being the first conference in Australian and beyond to make available papers online for the past decade with more than a million readers of almost 700 papers to its credit. From its humble beginning in 1998, the conference has become truly international, attracting presenters from many coal producing countries and mining machinery manufacturers including, Canada, Czech Republic, China, France, Germany, India, Iran, Italy, Japan, Kazakhstan, New Zealand, Poland, south Africa, Russia, Ukraine, UK, USA, and Turkey. Papers are presented by academics, researchers, mine operator engineers, geologists, consultants and equipment manufacturers. The cover of each proceeding in this series is shown on the following page.

The period of uncertainty in the future of coal mining and its export has cast some gloom on the picture in the past year. Many mines changed hands and some worked at reduced hours with temporarily production cuts.

However, the mining industry in Australia still remains viable and will remain an exporter of coal for many years to come. There were many mines facing difficulties, however they rearranged their operations, with some companies ended up selling some of their mines, while other companies dug their heals in, but with reduced operational enthusiasm. As a result we see many engineers and geologists move away to different professions, thus affecting the innovation capability of their operations.

In this proceedings there are 37 papers addressing a wide range of topics; mining methods, both surface and underground, mine transport, mine exploration and mine gas drilling, ground control, coal outburst and rock bursts, mine safety, mine fires and control, spontaneous combustion, mine automation, mine management and logistics. We would like to acknowledge the support we have received in sponsorship, and logistics from various organisations and their names will be appropriately placed on record in more than one way.

Special thanks to the editorial panel members and the paper reviewers; Johlene Morrison for typesetting the conference proceeding, the creation of the conference website and overseeing the workshop logistics and efficient administration of the conference website, Kevin Marston and Shahin Aziz for conference general management, the University of Wollongong printery staff Terry Campani for designing the conference proceedings cover page, Garry Piggott and his colleagues for printing the conference proceedings and Gypsy Jones café for catering. 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 online 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 COVER
AUSTRALIAN COAL INDUSTRY
COMPETITIVENESS ASSESSMENT

Paul Hodgson¹, Francis Norman²

ABSTRACT: National Energy Resources Australia (NERA), in association with Accenture, have completed the Australian Coal Industry Competitiveness Assessment (ICA), including an Industry Competitiveness Framework (ICF) and Industry Competitiveness Score (ICS). The score provides NERA with a data-driven analysis of how to effectively allocate and direct their resources to deliver maximum industry impact. It also delivers a baseline against which the industry can measure its performance in future releases. This report outlines the methodology utilised and the results and insights gained from the ICS.

The ICS provides a comprehensive, data-driven assessment of the Australian coal industry from a global viewpoint. The results identify numerous areas for more rigorous study and suggest a number of innovative and collaborative improvements that, if implemented, will have a significant impact on overall industry competitiveness. NERA has a role in helping to increase engagement across the industry on a national level to ultimately deliver greater value for the nation. In future years, the ICS will provide a solid baseline against which the industry can measure improvement.

INTRODUCTION

From the baseline results, Australia’s black coal industry has an overall competitiveness score of 5.8 out of 10, placing the nation 3rd on the leader board of global peers, marginally above the world average of 5.4, and lagging behind the world best, China, at 6.5. Modelling and analysis finds that improvements across several priority areas have the ability to improve the country's competitiveness score by 18 per cent, and increase the value added to the Australian economy by A$4.5b.

Australia performs poorly across the exploration and development and extraction and production phases of the value chain, resulting in a mediocre standing for the country amongst the peer group. With the world undergoing an energy generation transition, moving away from fossil fuels to cleaner energy sources, the competition between energy sources is set to increase. Success for the Australian coal industry lies in performing consistently well across all eight pillars of competitiveness and, ensuring the energy generation and steel making demands of the developing world are met with Australia’s high-quality coal.

To achieve improvements in Australia’s overall industry competitiveness, this report has identified five priority areas where changes in the short term have the ability to affect the country’s performance:

- **Supply chain:** Supporting the industry by setting up regional supply hubs, and coordinating key activities across the industry to increase standardisation and utilisation rates.
- **Research and innovation:** Resolving the gap between research and commercialisation, and investing in the “connected mine”, with the goal of increasing overall productivity and lowering costs.
- **Workforce:** Upskilling the workforce to be competent in the “new” way of working, and investing in local capability for the closure and rehabilitation phase to maximise potential value and increase workforce ability.
- **Regulatory reform:** Shifting the interaction between government bodies and industry to a partnership-based model, establishing clear requirements and regulations, and developing greater transparency on the tax and royalty system, to reduce red tape costs and increase cross-stakeholder collaboration.

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¹ General Manager Innovation and Stakeholder Engagement (East Coast), National Energy Resources Australia
² General Manager Innovation and Strategy, National Energy Resources Australia
Social License: Building a consistent message for the Australian public, with a focus on growing mining and energy literacy, and articulating the important role Australia’s coal industry plays in the global energy transition; and also, focusing on rehabilitation of heritage sites.

AUSTRALIA AND THE GLOBAL COAL MARKET

Given the export nature of Australia’s coal industry, the global coal market has a significant influence on the country’s competitive position. As of 2015, Australia had a 36 per cent share of the world’s coal exports (26 per cent thermal and 63 per cent metallurgical). Figure 1 provides a view of the supply and demand structure of the global coal market. Asia fuels the majority of global demand and even though they are major producers, both east and west Asia are significant importers of coal. Australia’s large potential production volume, high quality, reserves, reliability of supply and unique location creates a structural advantage compared to other major exporting nations.

Australia is well positioned geographically to meet the significant Asian coal demand

Figure 1: Global Black Coal Market

Figure 2 breaks down Australia’s black coal industry across the four producing states, clearly illustrating strengths and weaknesses of the local industry. Coal production in Australia is concentrated within Queensland (QLD) and New South Wales (NSW), with over 97 per cent of the country’s black coal production occurring in the two states (Wood Mackenzie, 2015). The significant brown coal production and reserves concentrated in Victoria is not covered in this study. The tight clusters of mining operations create a critical mass of infrastructure and suppliers that is vital for a cost competitive exporting industry.

Across all four states, mining and preparation costs are well above the world average (Wood Mackenzie, 2015). This contributes to the poor performance in the Extraction and Production phase of the value chain. Fortunately, QLD and NSW have high quality product and sizeable reserves. This combination positions the country as a key supplier of world coal, now and in the future.
Australia's high quality and sizeable coal reserves combined with concentrated operations outweigh the industry's low-cost competitiveness

Figure 2: Australian black coal market

The global coal industry produces two key products, thermal and metallurgical coal. While similar, they have fundamentally different applications, affecting market dynamics. Thermal coal is used to produce energy, while metallurgical coal is a key input into the production of steel.

The world is undergoing an energy generation transition, moving away from fossil fuels to more environmentally sustainable sources of energy. World energy consumption share of coal is projected to decline by 3.5 per cent over the next 15 years; however, total coal consumption is still expected to grow by 419 Mtoe during this same period. This is illustrated in Figure 3.

Due to rising populations and economic growth, demand for coal fired power is expected to grow in developing nations; India, in particular, will see electricity requirements double by 2040 (IEA, 2015).

With aggressive environmental targets, it is expected that future demand will shift to high-efficiency, low-emissions coal-fired power, requiring high quality inputs. Against these conflicting trends, thermal coal is forecast to grow 8.5 per cent by 2030 (EIA, 2015). While still positive, this growth is significantly lower than the last 15 years as shown in Figure 4.
Over the last three decades, China has experienced an unprecedented level of urbanisation. The resulting economic growth has seen global steel production grow by 75 per cent (5.5 per cent p.a.) over the last 15 years alone (WorldSteel, 2010, 2014). This was the key contributor to the enormous growth in Australia’s coal industry over the same period.

As China’s economy matures, growth has begun to slow, resulting in the softer steel demand shown in Figure 5. However, due to economic growth throughout the rest of Asia, metallurgical coal will still be in demand. The IEA and the World Steel Association expect steel production to continue growing at 2 per cent p.a. until 2030, which is slower than the previous 15 years (IEA, 2015; Aurizon, 2015). India is set to be the largest source of this growth, however the country has very few high quality metallurgical coal reserves (Aurizon, 2015), so any major increase in demand will have to be met through imports, placing Australia, and Queensland in particular, in a favourable position. Figure 6 shows the industry competitiveness results and insights.
Steel demand is expected to grow steadily in India while, demand in China will plateau.

**Figure 5: Global steel production 2000-2030**
*Source: IEA 2015, World Steel Association*

**INDUSTRY COMPETITIVENESS RESULTS AND INSIGHTS**

**Industry Competitiveness Score**

<table>
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<th>Industry Competitiveness</th>
<th>Industry Growth Enablers</th>
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<td><strong>5.8</strong></td>
<td><strong>Supply Chain</strong></td>
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<tr>
<td>World Best</td>
<td>6.4</td>
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<tr>
<td>China - 6.5</td>
<td><strong>Worldforce</strong></td>
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<td></td>
<td>5.2</td>
</tr>
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<td></td>
<td><strong>Research &amp; Innovation</strong></td>
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<td>7.1</td>
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<td></td>
<td><strong>Government &amp; Public</strong></td>
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**Figure 6: Industrial competitiveness dashboard**

From the analysis completed, Australia has an Industry Competitiveness Score (ICS) of 5.8 out of 10, behind the world best, China, while marginally exceeding the world average of 5.4. The country performs strongly in the coal transportation phase of the value chain, with a score of 8.4, and also performs better than the world average in three of the four Industry growth enablers. However, weak results in both the exploration and development and the extraction and production phases ultimately undermine the country’s overall competitiveness.
China comes out on top of the ICS due to having one of the lowest costs across the value chain. China is the largest consumer and producer of coal in the world. The country performs the best in the exploration and development and coal transportation phases, and ranks third in the extraction and production phase.

Table 1 presents the ICS leader board, where Australia ranks as the world’s third most competitive coal producing nation. While the country ranks only slightly above average, the spread of scores across the peer group is low.

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<th>Country</th>
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<td>China</td>
<td>1</td>
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<tr>
<td>South Africa</td>
<td>2</td>
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<tr>
<td><strong>Australia</strong></td>
<td>3</td>
</tr>
<tr>
<td>United States</td>
<td>4</td>
</tr>
<tr>
<td>Russia</td>
<td>5</td>
</tr>
<tr>
<td>Indonesia</td>
<td>6</td>
</tr>
<tr>
<td>Colombia</td>
<td>7</td>
</tr>
<tr>
<td>Canada</td>
<td>8</td>
</tr>
<tr>
<td>Vietnam</td>
<td>9</td>
</tr>
<tr>
<td>Mozambique</td>
<td>10</td>
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Inspection of the results suggest this is because no country performs consistently well across all eight pillars of competitiveness. For example, Canada, ranked 8th, scores very highly in the industry growth enabler pillars (i.e. supply chain, research and innovation, workforce, and government and public involvement), however, it is among the worst performers in the extraction and production and coal transportation pillars. These results suggest all 10 countries within the peer group have significant room for improvement and that, with industry commitment and policy support, Australia has the opportunity to significantly increase its competitiveness standing.

**EXPLORATION AND DEVELOPMENT**

Australia performs poorly in the exploration and development phase of the industry value chain with a score of 4.7, only slightly above the world average of 4.5, and trailing behind the world’s best, China, with a score of 6.3.

Exploration spending has fallen significantly since its peak in 2011 (SNL, 2016; ABS, 2016); the decrease is in line with exploration spending across the world, which is down 72 per cent over the same period (SNL, 2016). Subdued coal prices and a slower demand growth are clear contributors to this fall in spending (Index Mundi, 2016). Figure 7 shows the strong correlation between exploration spending and the price of thermal coal over the last seven years, both in Australia and across the world. This trend is unlikely to reverse in the short term given the uncertainty surrounding coal prices.

For Australia’s coal exploration sector to flourish when coal prices rebound, it is essential to create an environment where regulation and costs promote, rather than hinder, an active exploration sector. According to the 2015 Fraser Institute Survey of Mining Companies, 55 per cent of respondents in Queensland and New South Wales reported that regulation uncertainty had a negative impact on the states’ investment attractiveness, versus only 13 per cent in Western Australia (Fraser Institute, 2016). The industry must look to other geographies and industries that are promoting exploration more effectively.
Australia must make progress in creating an attractive exploration environment for new and existing firms. If the country cannot develop and operate projects competitively, the ability to find new reserves is inconsequential.

Over the last decade, coal prices were pushed well above their long-term average, fuelled primarily by demand from Asia. In spite of Australia’s uncompetitive development capability and poor regulatory environment, there was significant expansionary capital expenditure which saw coal production grow by 124 Mt (Wood Mackenzie, 2015). However, as prices have weakened in the past 3 years, the industry’s expansionary capital expenditure has followed (Wood Mackenzie, 2015; Index Mundi, 2016). Figure 8 shows the downward trend in Australia’s CAPEX to production ratio over this period.

Following a boom in production growth, capital expenditure in Australia has fallen sharply

Development is a key weakness for the Australian coal industry. Capital costs for projects built over the last 5 years averaged US$7.2/t, the highest in the world, and almost 50 per cent above average (Wood Mackenzie, 2015; Accenture). While excessive demand during the boom saw significant cost inflation and project delays, this does not fully explain Australia’s poor performance; instead, structural factors; such as the high cost of labour, are a major cause of this weakness. In the past two years, construction and labour costs have been falling; however, they are still among the highest in the world, and further labour cost reductions are unlikely to provide the step change in costs required. The country’s current poor development capability is a severe barrier to investment. If the industry is to approve major new projects, investment must be made into new and innovate ways to compete.
The combination of a complex regulatory environment (State and Federal Government) and low social licence to operate creates an unfavourable environment for Australian coal companies to venture into new coal projects. A recent study by the World Bank found 50 per cent of respondents believe the coal industry does not benefit their local communities and, 65 per cent believe it is having a negative impact on the local environment. The lack of a clear social licence for the coal industry is a significant impediment to new operations. To improve social licence, the industry should start by building a clear and consistent message targeted to the Australian public, increasing energy literacy and articulating the need for high quality coal in the global energy transition.

Due to Australia’s poor development capability, based on current projections, only four new mines have a high probability of opening in the next decade (Wood Mackenzie, 2015). If additional greenfield projects are to become operational, the country must find ways to reduce upfront capital costs to be more in-line with the rest of the world, and work to improve its current social licence position.

EXTRACTION AND PRODUCTION

Australia performs poorly overall in the extraction and production phase of the industry value chain with a score of 5.0, below the world average of 5.3, and trailing behind the world’s best, Russia, with a score of 6.7.

Regardless of mine type (surface or underground) or coal product (metallurgical or thermal), Australia performs below average in both mining and coal preparation in the extraction and production phase. Australia’s average mining and coal preparation costs are US$37.50/t and US$6.00/t respectively. This is substantially higher than the peer group average of US$29.60/t and US$3.10/t respectively, indicating there is significant room for improvement.

Over the last few years, foreign exchange factors have played a significant part in pushing down operating costs for developing nations. For example, in Russia, the devaluation of the Rouble has reduced operating costs significantly, making their industry more competitive. While Australia’s costs are significantly higher, 75.3 per cent of coalmines were able to generate positive margins in 2015. This view on overall margin is illustrated in Figure 9. Metallurgical coalmines perform better and have a higher share of mines with positive margins compared to thermal coalmines (87.6 per cent compared to 65.9 per cent respectively) (Wood Mackenzie, 2015; Accenture).

Although this paints a positive picture of Australia’s coal industry, many mines are operating close to the breakeven point. Given the volatile nature of coal prices, the profitability of Australia’s coal industry can move quickly. Scenario modelling suggests that, based on 2015 costs, a fall in the price of coal of only 15 per cent would see 49 per cent of mines operating with negative margins. On the other hand, should prices rise 15 per cent, based on 2015 costs, only 15 per cent of Australian mines would be operating on negative margins.

![Figure 9: Variations in Australian coal mines profitability margins](image-url)
Cost cutting initiatives, since prices began to fall, have kept Australia's coal mines competitive up until this point. However, progress is beginning to slow, particularly as automatic cost stabilisers such as the falling Australian dollar have taken effect, and quick savings such as workforce reductions have already been implemented. Now the industry is faced with the much tougher task of tackling the structural factors contributing to Australia's poor cost competitiveness. If not addressed as a priority, high-cost and low-margin mines in Australia will be forced to close prematurely.

A key structural factor contributing to the country's high mining and coal preparation costs is the workforce. Education and training levels scored very highly in Australia compared to the peer group, which contributes to the country having the highest salaries in the world (approximately 30 per cent higher than the US) (Hays, 2016). While salaries have peaked and Australia has seen a marked decrease, this is in line with the rest of the world, and as such has not improved the country's competitive position.

Although salaries are high in Australia, labour productivity, measured as marketable (product) tonnes per employee, is also high, ranking second overall as shown in Figure 10 (Wood Mackenzie, 2015). Australia scores highly due to the nation having more established and advanced operating environments compared to other countries in the peer group. Australia can further invest in increasing its current use of technology and automation in mining operations, for example, by learning from and considering remote operations and driverless trucks used in the metals mining sector. This will drive further productivity gains, offsetting the uncompetitive labour cost to improve overall competitiveness.

The ability to produce high quality coal at scale is key to the competitiveness of Australia's export based coal industry. Metallurgical and thermal reserves in Queensland and New South Wales are among the largest and highest quality in the world. The ability to export this coal at scale sets Australia apart from many of its competitors. For example, Mozambique exports high quality coal, however the total production of the country is only one per cent of Australia's output (Wood Mackenzie, 2015).

Extraction and production is the weakest phase in the value chain for Australia's coal industry. While the country has significant high quality metallurgical and thermal coal operations, it cannot currently mine and process this coal competitively. By tackling structural disadvantages, the industry has the opportunity to make a substantial impact on overall competitiveness.
COAL TRANSPORT

Australia performs well in the coal transport phase of the industry value chain with a score of 8.4, well above the world average of 6.7, and only just behind the world’s best, China, with a score of 8.6.

The transport of coal is a significant cost factor for industry operators, representing 25 per cent – 40 per cent of the cost for seaborne coal (Woods Mackenzie, 2015; Metalytics, 2015). Given Australia exports 88 per cent of its coal production, strong performance here is crucial in the overall competitiveness of the sector. Fortunately for the country, geography plays a major role in determining a coal producer’s transportation competitiveness. Shorter distances to ports, and to final markets, significantly reduce the infrastructure required and the total cost of transportation.

Coal production in Australia is concentrated within Queensland and New South Wales, with over 97 per cent of the country’s black coal production occurring in those two states (Woods Mackenzie, 2015). The large clusters of mining operations and maturity of infrastructure mean that most mines operate within proximity to world-class port and rail infrastructure, as shown in Figure 1. As a result, Australia’s average distance from mine to port of 206 km is among the lowest in the world (Metalytics, 2015).

The total inland coal transport task in Australia is estimated at 88.9 billion tonne kilometers, of which rail accounts for 95.9 per cent (BITRE, 2006). Over 4 years from 2008 to 2012, fuelled by export demand from Asian markets, Australia invested heavily in rail infrastructure, boosting capacity by 120 Mt (35 per cent) (Woods Mackenzie, 2015).

During the industry’s rush to add capacity, transportation costs soared. A concerted effort has since been made to reduce costs, in particular, a significant focus has been placed on improving productivity and utilisation of these assets through regulated collaboration and coordination. Across the country, utilisation rates have reached 75 per cent, and costs have fallen by 38 per cent as seen in Figure 12 (Woods Mackenzie, 2015).
While the location of Australia’s current coal operations is favourable to land based transport costs, this may change. The next major untapped reserves of coal are in the Galilee and Surat basins, located over 500km inland, with no significant rail infrastructure. Adani estimates it would cost A$2.2bn to build the significant rail infrastructure needed to export from the Galilee basin (Qld Department of State Development, 2016). This has been a major impediment to any development within the region.

Successful transportation capability, for an exporting nation, requires quality port infrastructure. Exports from Australia’s seven largest ports have grown an average of 5.6 per cent p.a. over the last decade (Woods Mackenzie, 2015). To support this growth, the industry has added an additional 246 Mtpa of world class export handling capacity since 2006 (Woods Mackenzie, 2015). Newcastle port is now the largest coal export terminal in the world, exporting 158.1 Mt of coal in 2014-15 (BITRE, 2006). As a result of high quality infrastructure and having the world’s leading port utilisation rate of 75 per cent, Australia’s port costs are among the lowest in the world, adding on average, only US$3.6 per tonne shipped (Woods Mackenzie, 2015; Metalytics, 2015).

Shipping is a major cost to the seaborne coal market; however, distance is the major driver of costs, providing little opportunity for the industry to improve competitiveness. Fortunately for Australia, the world’s four largest coal importers; China, Japan, India and Korea, are all within close proximity, leading to lower shipping costs, as illustrated in Figure 13.

The combination of quality port and rail infrastructure, along with short distances mean Australia’s cost per tonne for land based transportation of US$7.1/t is the third lowest among the peer group (Woods Mackenzie, 2015; Metalytics, 2015). The industry must continue to work collaboratively across shared transportation infrastructure, both rail and ports, if it is to remain competitive.

CLOSURE AND REHABILITATION

Closure and rehabilitation is a crucial phase of the industry value chain, as it leaves a significant legacy, affecting local communities and the environment. Due to the complex and different nature of the physical environment, political landscapes, and social culture across the peer group, an overall score was not developed. Instead, this study exclusively examines Australia’s mine closure costs and social and community engagement indexes.

As of 2015, there were 95 coalmines operating across Australia (Woods Mackenzie, 2015). Figure 14 shows the forecast drop in coal production should no new mines be opened. This clearly indicates the enormous closure and rehabilitation activity that is coming. Mine closure and rehabilitation is both complex and expensive. Rehabilitation covers a range of activities including establishing final land form position, revegetation and ongoing environmental
monitoring. As a result, rehabilitation is significantly more expensive and requires a longer duration to complete compared to mine closure activities. The estimated liability for mine closure activities such as decommissioning based on currently operating mines within Australia, over the next 30 years stands at US$3.4 Billion (Woods Mackenzie, 2015). This indicates the substantial overall cost to the coal industry for both closure and rehabilitation.

![Australia's shipping and port costs compare favourably against its major seaborne exporting peers](image)


**Figure 13: Port and shipping costs**

![Mine closure and rehabilitation activity is set to increase significantly over the next 30 years as current operating mines begin to close](image)

Source: Wood Mackenzie Ltd. Q4 2015

**Figure 14: Production outlook from operating mines in Australia**

Issues in the sector arise when sufficient funds have not been allocated to adequately complete mine closure. While many companies plan for the rehabilitation, it is vital that sufficient funds are set aside well in advance of the cost. Where necessary, the government must step in to ensure state money is not ultimately required. In tandem, the industry must find ways to reduce this future liability. One way to reduce this liability is for operators to progressively rehabilitate mining areas which are no longer used for mining purposes. Australia should collaborate with other countries to train its workforce to be able to operate competently during mine closure and ongoing monitoring, as well as leverage the opportunity to learn lessons and gain insights from countries that have experience in this phase of the value chain (e.g. UK).
It is crucial that eventual mine closure and rehabilitation is carried out successfully as the impact to the regions and communities will be lasting and significant. Failure to perform these activities properly on just one coalmine could trigger significant backlash from the community, and affect the overall credibility and social licence to operate for the entire coal industry.

The amount of socio-economic contribution and involvement by the coal industry in Australia ranks as the highest amongst the peer group. Figure 15 shows that Australia’s contribution is more than double that of countries such as Colombia, Indonesia, Mozambique and South Africa (Fraser Institute, 2016).

![Image: Coal industry contribution to community against public perception and support](source)

**Figure 15: Coal industry contribution to community against public perception and support**

However, even with the highest investment into local content development, the general public perception toward the coal industry in Australia is very poor, with 46 per cent of the public surveyed having a negative view (World Bank, 2014). Figure 16 shows that investing more does not directly translate to having a greater share of buy-in from the public. In Australia’s case, it shows that investments made have not been effective and there is a potential disconnect between what the coal industry has been doing to engage and address the concerns of the public.

The industry must listen to the public and look to address concerns with projects that generate tangible benefits to the community and the environment. One practical example is by revisiting heritage mines and ensuring they are properly rehabilitated, and fit for use by future generations. An undertaking like this, which would require significant collaboration from the industry and its partners, would start to rebuild the industry’s social license. A focus on building stronger collaboration with government bodies, and changing the current engagement model from being an ‘enforcer of rules and regulations’ to being a part of the process would also lead to overall benefit to the industry and Australian community.

There is time for the coal industry in Australia to prepare and position itself to effectively manage the closure and rehabilitation phase of the industry value chain. *By increasing collaboration with various stakeholder bodies, positioning and skilling workforce accordingly, and learning lessons from other countries, Australia has the opportunity to increase its competitiveness and maximise value for the industry and country in this phase of the value chain.*

ACKNOWLEDGMENTS

The results and findings from this report are based on research conducted over the course of ten weeks from October to December 2016. The objective was to create an industry relevant measure of the coal industry competitiveness that was robust and repeatable, allowing
improvements to be tracked over future releases. The assistance of Accenture in undertaking this work with NERA is acknowledged.

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MAKING EFFECTIVENESS AUDITS TRULY EFFECTIVE

Paul Harrison¹, Phil Goode²

ABSTRACT: The Queensland Coal Mining Safety and Health Act 1999 and the Mining and Quarrying Safety and Health Act 1999 require a mine to have a Safety and Health Management System (SHMS) in place to manage the risk to safety and health of persons at the mine. The legislation assigns the obligation to the site senior executive to develop and implement the mine’s SHMS. It assigns a further obligation to the mine operator to audit the effectiveness of the system put in place by the site senior executive.

What is exactly meant in the legislation by the term effectiveness, and how to go about assessing effectiveness has been the topic of much debate since the legislation was enacted. In 2008, the Queensland Mines Inspectorate provided some clarification on the issue, when it published Queensland Guidance Note QGN09 Reviewing the Effectiveness of Safety and Health Management Systems. However, QGN09 is not an exhaustive treatment of reviewing the effectiveness of an SHMS.

In this paper, former Queensland Commissioner for Mine Safety and Health, Paul Harrison, and former Queensland Chief Inspector of Mines, Phil Goode, discuss effectiveness audits from the perspective of the authors’ experience and propose a tool for quantitative measurement of SHMS effectiveness.

INTRODUCTION

The mining industry in Queensland today enjoys one of the most enviable safety records of anywhere in the world, but this was not always the case. Prior to 1994, the record was nothing to boast about. In the 20 years prior to 1994, there was an average of 5.3 fatalities per year in Queensland mines. In the late 80s/early 90s, one large hard rock mine in Queensland recorded 9 separate fatalities in a period of just over 2 years. By the early 1990s the rate of reduction, since 1950, in the Lost Time Injury Frequency Rates (LTIFR) had also started to slow.

These trends were causing concern to industry and the Queensland Mines Inspectorate. In 1992 a legislative review group was established to update the legislation. Progress with the legislative review was initially slow, however, the underground explosion at Moura No. 2 coalmine in Central Queensland in 1994 was the watershed moment in mining history that changed all that.

Following this disaster, the management of mining safety and health in Queensland was completely overhauled. In 2001 new legislation was introduced using a risk based approach to cope with the hazards at a mine and required the development of mine site Safety and Health Management Systems (SHMS). This approach soon gained momentum across the Australian mining industry as a whole.

Prior to 2001 there were no published standards available for SHMS, but in 1997 the Queensland Mines Inspectorate developed their own approach based on the international quality management systems standard ISO 9001:1994 (International Organization for Standardization 1994). They called it SafeGuard, and it was designed to help mines establish and assess their SHMS, to measure its performance and to help ensure continual improvement of the system. Of course, today there are Australian and international standards that specifically address SHMS, namely Australian Standard AS/NZS 4801:2001 (Standards Australia 2001), OHSAS 18001:2007 (BSI 2007) and Draft International Standard ISO/DIS 45001:2016 (International Organization for Standardization 2016).

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The new approach to managing hazards at mine sites has proved effective, as evidenced by the marked improvement in incident statistics (refer to Figure 1 and Figure 2 for examples – data provided courtesy of the Queensland Department of Natural Resources and Mines).

The Queensland legislation requires the senior manager at a mine site to establish the mine’s Safety and Health Management System and requires the mine operator to periodically audit the system to ensure it is, and remains, effective. Colloquially, these have come to be called operator effectiveness audits.

![Figure 1: Lost time injury rate comparison for Queensland and NSW coal mines post Moura No. 2 disaster](image1)

![Figure 2: Annual fatality rate for Queensland mines and quarries](image2)

**MANAGEMENT SYSTEM EFFECTIVENESS**

Queensland Guidance Note QGN09

What is exactly meant in the legislation by the term *effectiveness*, and how to go about assessing effectiveness has been the topic of much debate since the legislation was enacted. The legislation merely states that the mine operator has an obligation to *‘audit and review the effectiveness and implementation of the safety and health management system to ensure the*
risk to persons from coal mining operations\(^1\) / operations\(^2\) is at an acceptable level (Section 41(1)(f) Coal Mining Safety and Health Act 1999 and Section 38(1)(e) Mining and Quarrying Safety and Health Act 1999). It provides no further direction on, or clarification of, the requirement.

In 2008, the Queensland Mines Inspectorate provided a degree of clarification when it published Queensland Guidance Note QGN09 Reviewing the Effectiveness of Safety and Health Management Systems (Department of Mines and Energy 2008). While providing additional guidance, QGN09 is not an exhaustive treatment of reviewing the effectiveness of an SHMS (Department of Mines and Energy 2008, p.6). It identifies some of the key subsystems that should be included in an effectiveness audit. There are other SHMS elements such as planning, objectives and targets, emergency response and document and record control that it does not call direct attention to.

**Management system standards’ perspective**

AS 4801:2001 (Standards Australia 2001, p.12) states that, ‘the organization shall establish, implement and maintain an audit program and procedures for periodic OHSMS audits to be carried out by a competent person, in order to determine whether the OHSMS is effective in meeting the organization’s policy as well as objectives and targets for continual OHS improvement.

ISO/DIS 45001:2016 (International Organization for Standardization 2016, p.22) states that an organisation must establish an audit program that, amongst several other requirements, provides, ‘conclusions on the continuing suitability, adequacy and effectiveness of the OHanS [ occupational health and safety] management system’.

The word effective or variations of it appears 28 times in AS 4801:2001 and 39 times in ISO/DIS 45001:2016. Thus, it is clear that routine ongoing assessment of management system effectiveness is a key requirement for an SHMS. The requirement in the Queensland mining safety and health legislation does not add any new dimensions to the audit process. Any management system audit must be structured to provide for an assessment of effectiveness.

**What is management system effectiveness?**

In the context of QGN09, effective means that the SHMS reduces the level of risk to safety and health of persons affected by the operation of a mine to within acceptable limits and as low as reasonably achievable. It is considered that achieving this goal will result in continual improvement of safety and health standards and performance.

According to International Standard ISO/DIS 45001:2016 (International Organization for Standardization 2016), the effectiveness of a management system is the extent to which it delivers on planned activities and planned organisational objectives. In the case of an SHMS, these activities and objectives relate to the mitigation of safety and health risks as per QGN09.

An audit of the effectiveness of an SHMS, in its simplest terms, is an assessment of:

The adequacy and suitability of the measures used to set and monitor organisational safety and health objectives.

The extent to which planned activities have been implemented and organisational objectives have been realised. This accounts for how well the system is designed for mitigating safety and health risks and how well it is performing in that respect.

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\(^1\) coal mining operations - defined in the Coal Mining Safety and Health Act 1999 Schedule 3 Dictionary coal mining operations

\(^2\) operations – defined in the Mining and Quarrying Safety and Health Act 1999 Section 10 Meaning of operations
THE EXPERIENCE

The Queensland mine safety and health regulator have been engaged in a debate about what constituted management system effectiveness and how to undertake an objective assessment of it.

One criticism of operator’s effectiveness audits was that they tended to be more desktop system audits or compliance audits rather than audits assessing how well the systems are implemented at worker level. An SHMS can only be effective if it is fully implemented. Another criticism that arose was the frequency at which the audits were done (i.e. not frequent enough). Consequently, the standard of operator effectiveness audits was variable. Some other issues observed included:

- Failures to gather sufficient representative evidence to make a reliable and objective assessment
- Insufficient time and resources allocated to the audit to carry out an adequate assessment of what are large and complex systems, especially in relation to assessing implementation of the system at the worker level (e.g. audits conducted in two or three days by a one-person audit team – consider the 13 element management system given as an example in Figure 3, a three day, one person audit, deducting time for entry meeting, exit meeting and initial draft report, would only allow an hour and a quarter to gather and assess data for each management system element)
- Lack of a quantitative measure of system effectiveness to gauge and compare the SHMS performance.

Figure 3: Illustration of how elements of an SHMS fit into the PDCA framework

KEY CONSIDERATIONS FOR ASSESSING EFFECTIVENESS

Continual improvement
Management system standards are built on a management model referred to as the Deming Cycle. The Deming Cycle is a model for the control and continual improvement of processes, products and services. The model has four steps – Plan, Do, Check, Act (PDCA). PDCA requires an organisation to think about:

- What it wants to achieve
- Planning realistically to do it
- Allocating appropriate resources and ownership
- Putting the necessary tools in place and training people to use them
PDCA describes a model for continual improvement of the effectiveness of a management system. It is achieved through the use of policy, system objectives, system implementation, system monitoring, audit and review, followed by corrective and preventive/improvement action. Any SHMS must be structured to fit within the PDCA framework. Figure 3 illustrates the PDCA framework as it applies to an SHMS with 13 main elements.

AS 4801:2001, ISO/DIS 45001:2016 and QGN09 all place a lot of emphasis on continual improvement as it is the cornerstone of any management system. Hence, a critical factor in the assessment of management system effectiveness is the extent to which continual improvement is embedded in the organisation. The tools used for SHMS effectiveness audits described in this paper have been developed using this model as the basis.

**Trigger events**

There are many events that may trigger the requirement for changes to the SHMS which form part of the process of continual improvement. Some examples include:

- Incident investigations and hazard reports
- Non-conformance reports and corrective action requests
- Audits, inspections and observations
- Toolbox meetings, safety and health committee meetings
- Management review
- Significant changes in operations
- Regulatory action (e.g. mine record entries, directives, prosecutions, improvement notices)
- Safety bulletins and alerts (internal and external)
- Other external factors (e.g. legislative changes, Level 1 exercises, Coronial Inquiries).

When auditing the effectiveness of an SHMS it is important to assess how changes resulting from such trigger events are managed to provide continual and sustainable improvement (i.e. an effective change management process is in place). Figure 4 illustrates a continual improvement cycle arising from trigger events and the management of change process.
Since management of change is a critical element of the continual improvement cycle, an audit of management system effectiveness must include an assessment of how trigger events that could potentially require changes to the SHMS are handled.

Management review

AS 4801:2001 (Standards Australia 2001, p.12) states that, ‘the organization’s top management shall, at intervals that it determines, review the OHSMS [occupational health and safety management system], to ensure its continuing suitability, adequacy and effectiveness. The management review process shall ensure that the necessary information is collected to allow management to carry out this evaluation.’

ISO/DIS 45001:2016 (International Organization for Standardization 2016, p.47) states that, ‘management reviews are a critical part of the continual improvement of the management system. The purpose of these reviews is for top management to undertake a strategic and critical evaluation of the performance of the management system, and to recommend improvements.’ This is often an area that is not well implemented and, hence, deserves particular attention during an audit of management system effectiveness.

Management reviews should consider:

- Suitability: the extent to which the management system fits and is right for the organisation’s purpose, operations, culture and business systems
- Adequacy: the extent to which the management system is sufficient in meeting the applicable requirements (includes the training and competency of workers)
- Effectiveness: the extent to which planned activities are realised and planned results achieved.’

The purpose of management reviews is to assess system performance and identify opportunities for continual improvement. They should include an evaluation of how well the SHMS is integrated with other business processes and the strategic direction of the organisation.

Representative sampling

Like any other activity, audits are constrained by cost, time and resources. An SHMS can be very large and complex, and associated records extensive. Thus, it is not possible to evaluate every system document or record, or interview every worker. It is only possible to evaluate a sample, but that sample should be representative of the whole system. This must include sampling of evidence about how the SHMS is managing the risk of hazards, particularly principal or critical hazards. An audit should verify how each element of the SHMS contributes to the management of a particular hazard.

Collecting objective evidence to support findings of conformance and non-conformance using accepted sampling methods helps to minimise auditor bias. Findings are based on objective evidence not the subjective judgement of an auditor.

To ensure the sample selected for an audit is representative of the whole system, representative sampling methods must be used by the auditor when gathering data to test the effectiveness of the implemented system. Representative sampling using a probabilistic sampling approach will ensure that evaluations are objective, unbiased and consistent between auditors within the current audit team and with past and future audit teams.

Probabilistic sampling is broken into four types:

- Random sampling. The application of random sampling ensures that each member of the population being sampled has an equal chance of being selected. It selects samples purely by chance, allowing the sample sub-population to represent the entire population without bias. For this approach to sampling, random sample numbers are generated using random number tables or a random number generator. The latter can be achieved using an Excel spreadsheet.
• **Block sampling.** Block sampling is used when the population is very large and selecting a random sample would result in a sample set too large to manage. Block sampling examines a block(s) of contiguous items from within the population. A random number table or generator should be used to select the first sample in a block to avoid potential bias.

• **Stratification sampling.** Stratification sampling treats a population by dividing it into discrete sub-populations which have an identifying characteristic, for example:
  - Roster A vs Roster B
  - Day shift vs night shift
  - Permanent employees vs contractors.

The sub-populations are treated as separate populations. Sampling of the sub-populations allows the auditor to categorise the total population by sub-population, providing for an accurate comparison of one group against another as well as an accurate assessment of the total population.

Stratification sampling is useful when there are wide variations in the size or characteristics of a population. It reduces the variability of items within each sub-population and allows sample size to be reduced without increasing sampling risk.

*Interval sampling:* The purpose of interval sampling (also referred to as systemic selection) is to pick samples at various intervals. The number of sampling units in the population is divided by the sample size to give a sampling interval. For example, if the sample size is 30, a starting point within the first 30 items is selected at random, and then each 30th sampling unit thereafter is selected.

When using interval sampling, the auditor must take care that sampling units within the population are not structured in such a way that the sampling interval corresponds with a pattern in the population.

**Sample size**

Sample sizes can be determined either statistically or based on the exercise of professional judgement. The level of sampling risk that the auditor is willing to accept affects the sample size required. The lower the risk the auditor is willing to accept, the greater the sample size will need to be.

When circumstances are similar, the effect on sample size of factors such as an increase in the expected rate of deviation of the population to be tested will be similar regardless of whether a statistical or non-statistical approach to sampling is chosen.

**A QUANTITATIVE MEASUREMENT OF EFFECTIVENESS**

Quantitative measurement of system effectiveness can be achieved by evaluating the individual elements which make up the SHMS (examples of system elements are shown in Figure 3).

An audit tool specific to the organisation/site is developed prior to conducting the onsite component of the management systems audit, based on the documented management system structure. The purpose of the tool is to assess the level of conformance and/or non-conformance with performance standards for each of the management system elements. Representative sampling is undertaken during the onsite audit to obtain representative, unbiased and objective evidence of the degree of that conformance or non-conformance. Following the collection of evidence of conformance and/or non-conformance, each management system element is scored between 1 and 5 according to predetermined criteria. For example:

• **If no performance standard requirements have been considered, then the system element will score a 1**
If the performance standard requirements are demonstrated to be fully effective, then the system element will score a 5.

If implementation of the performance standard requirements is found to be somewhere in between (the most likely scenario), then the system element will score somewhere between 1 and 5, depending on the level of that implementation.

These scores can be categorised by traffic lights which give an indication of the extent to which system elements and processes satisfy the management system performance standards. Table 1 illustrates scoring against each element of the SHMS for example Mine X (these elements were introduced earlier in 3). Figure 5 demonstrate how the assessment can be represented visually as a gap analysis. This provides a quantitative measure of management system effectiveness which can be used to benchmark performance over time and between sites/organisations.

Table 1: Example of measured effectiveness of Mine X’s SHMS

<table>
<thead>
<tr>
<th>Mine X SHMS element</th>
<th>Score</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Leadership and accountability</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>2. Legal and other requirements</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>3. Hazards and risk</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>4. Planning, goals and targets</td>
<td>3.6</td>
<td>70</td>
</tr>
<tr>
<td>5. Awareness, competence and behaviour</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>6. Communication and consultation</td>
<td>2.5</td>
<td>50</td>
</tr>
<tr>
<td>7. Design, construction and commissioning</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>8. Operations and maintenance</td>
<td>2.5</td>
<td>50</td>
</tr>
<tr>
<td>9. Documents and records</td>
<td>4.0</td>
<td>80</td>
</tr>
<tr>
<td>10. Suppliers, contractors and partners</td>
<td>2.5</td>
<td>50</td>
</tr>
<tr>
<td>11. Incidents and emergencies</td>
<td>3.0</td>
<td>60</td>
</tr>
<tr>
<td>12. Monitoring, audit and review</td>
<td>2.5</td>
<td>50</td>
</tr>
<tr>
<td>13. Management of change</td>
<td>2.5</td>
<td>50</td>
</tr>
</tbody>
</table>

Figure 5: Radar plot of SHMS effectiveness analysis for Mine X showing gap between measured state and target state

CONCLUSIONS

There is nothing special about an effectiveness audit. Australian and International Standards for SHMS, stipulate that all audits of the management system or its component parts must assess effectiveness. According to QGN09, effective means that the SHMS reduces the level of risk to safety and health of persons affected by the operation of a mine to within acceptable limits and as low as reasonably achievable. ISO 45001:2016 (International Organization for Standardization 2016, p.47) defines the effectiveness of a management system as, ‘...the

1 These are the results of an actual audit of a mineral mine.
extent to which it delivers on planned activities and planned organisational objectives’. Thus, an audit of the effectiveness of an SHMS, in simple terms, is an assessment of:

- The adequacy and suitability of the measures used to set and monitor organisational safety and health objectives
- The extent to which planned activities and organisational objectives have been realised.
- The cornerstone of the management systems approach is continual improvement, so for any system to be deemed fully effective, demonstrated evidence that continual improvement is fully embedded in business processes is required.
- A quantitative measurement of SHMS effectiveness is possible through the systematic use of representative, objective and unbiased statistical sampling of data about system performance, and the application of scoring criteria to the data gathered. This quantitative measure can be used to assess the current state of management system effectiveness against the target state, benchmark system performance over time and to compare sites/organisations.

ACKNOWLEDGMENTS

The authors would like to acknowledge the assistance of Earle Alexander, former Mines Inspector, one of the architects of SafeGuard and a lead auditor for nearly two decades. Earle’s advice and guidance on methodologies and tools for assessing management system effectiveness, which he has developed, honed and calibrated over 20 years and more than 100 audits has been invaluable.

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MICROSEISMIC MONITORING OF UNDERGROUND COAL MINES: OBJECTIVES, WARNINGS AND SENSOR ARRAY DESIGN

Richard Lynch

ABSTRACT: Microseismic monitoring is the only technique to provide 3D data to geotechnical engineers at underground mines and has become a standard tool in the deeper metalliferous and coal mines around the world. Objectives of the monitoring are typically rescue, prevention, control, warnings (which include identification of precursors for goafs and for large seismic events which may be associated with rock/coal-bursts) and back-analysis. Depending on the monitoring objectives, the required monitoring system sensitivity – the minimum moment magnitude above which seismic events are reliably recorded and quantified – would be between magnitude -2.0 and magnitude +1.0. While simple seismic activity – crack counting – is sometimes good enough for warnings of impending goaf occurrence, precursors of large seismic events associated with rock-bursts or coal-bursts are more difficult to identify. There have been some sporadic successes in metalliferous mines with high levels of seismic activity – and thus lots of large seismic events to calibrate against. The design of the seismic sensor array depends on the objectives, but typically involves sensors installed in the main and tail gates either side of the panel. Current best practice involves permanent installation into long up- and down-holes to achieve the 3D configuration required for reliable 3D location of seismic sources. A more cost-effective solution is to use temporarily-installed geophones along with a surface seismic station.

INTRODUCTION

Microseismic monitoring of mines is a technique routinely applied at over 300 mines around the world. It is the only technique capable of providing real-time 3D data on how the rock mass is responding to mining, and the field has matured considerably over the past 25 years [for example, see proceedings of the Rockbursts and Seismicity in Mines symposia from 1988-2017].

Routine passive microseismic monitoring of underground hard rock (metalliferous) mines has been common since the 1990’s [Mendecki, 1993] and has also been applied in the hard rock open pit environment [Lynch at el, 2005; Lynch and Malovichko, 2006; Meyer, 2015] and in the underground coal mine environment [Arabasz et al. 1997; Hatherly et al. 1997; Minney et al, 1997; Hayes, 2000]. Underground coalmines are different to underground hard rock mines in many aspects, which have significant implications for the design and operation of microseismic monitoring systems. Indeed, while some of the monitoring objectives are common, there are some objectives specific to coal mines – for example, using the microseismic data to warn of an impending goaf fall.

OBJECTIVES

In general, routine microseismic monitoring in underground coalmines facilitates the quantification of exposure to seismicity and provides data for efforts into prevention, control and prediction or warning of potential rockmass instabilities that could result in rock- or coal-bursts. The following specific objectives of monitoring the seismic rockmass response to mining can be defined [following Mendecki 1999 and 2001] in the following ways:

Rescue: To detect and locate dynamic rock mass instabilities, alert management to potential rock-related accidents and assist in possible rescue operations – including by monitoring of aftershocks.

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Prevention: To quantify the exposure to seismicity, confirm the rock mass stability-related design assumptions and to enable an audit of the particulars of a given design while mining. This assists in guiding of preventive measures, e.g. corrections to the designed layout, sequence of mining, rates of mining and support strategy.

Control: To detect spatio-temporal patterns of seismological parameters – for example an increase in the number of seismic events located on a stability pillar or geological feature, or an increase of the statistical seismic hazard, or changes of volumetric stress by analysis of ambient seismic noise – and relate them to the expected short to medium term behaviour within the volume of interest. This would facilitate and guide control measures, for example a change to the planned end position of a particular panel or temporary changes to personnel exposure. Also to integrate seismic event data into suitable 3D numerical stress models to enhance their predictive capabilities.

Warnings: To detect unexpected strong changes in the spatial and/or temporal behaviour of seismic parameters or certain defined characteristic patterns – for example increasing microseismic activity - that could lead to dynamic instabilities affecting working places immediately or in the short term. This would facilitate warnings to manage the exposure to potential goaf falls (and the possibly accompanying air-blasts), rock-bursts or coal-bursts.

Back-analysis: To improve the efficiency of both the design and the monitoring processes for stability of mine workings. Specifically important is thorough seismic and numerical modelling back-analysis of large instabilities even if they did not result in loss of life or in considerable damage. Back analysis of seismic rock mass behaviour associated with pillars, geological features and different mining rates, is an important tool in the quest for safer and more productive mining.

The following table contains the required seismic array capabilities to meet each of the standard objectives of monitoring.

<table>
<thead>
<tr>
<th>Objective</th>
<th>3D location error [m]</th>
<th>Minimum magnitude above which events consistently recorded</th>
<th>Typical inter-sensor spacing [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rescue</td>
<td>≤ 100</td>
<td>≤ 1.0</td>
<td>3000</td>
</tr>
<tr>
<td>Prevention and Back-Analysis</td>
<td>≤ 50-75</td>
<td>≤ 0.0 to 0.5</td>
<td>1000 – 1700</td>
</tr>
<tr>
<td>Control</td>
<td>≤ 15-20</td>
<td>≤ -1.0 to -0.5</td>
<td>350 – 600</td>
</tr>
<tr>
<td>Warnings</td>
<td>≤ 10</td>
<td>≤ -2.0 to -1.5</td>
<td>100 – 200</td>
</tr>
</tbody>
</table>

**WARNINGS**

Goaf falls

One of the main seismic hazards in underground coalmines is violent failure of the roof strata – goafing – and the possible attendant air blasts. Fortunately, in this environment the simple indicator of increasing microseismic activity is often correlated with goafing [de Beer, 2000]. In an early study at Moonee colliery in New South Wales, Australia [Edwards, 1998], it was found that warnings based on increased seismic activity resulted in false alarms 46% of the time. However, 76% of significant goafs and all of the major goafs were successfully forewarned in this manner. Warning times were between a few seconds and 150 minutes, averaging about 50 minutes, which is quite practical for mining operations.
A later analysis of the seismic and goaf data [Iannacchione, et.al., 2005] showed that increased seismic activity was a reliable indicator of impending goaf fall about 90% of the time at Moonnee. An example of a successfully predicted goaf is shown in Figure 1, and more examples of both successful and unsuccessful predictions are given in Figure 2.

![Figure 1](image1.png)

Figure 1: The cumulative number of accepted and located microseismic events vs time. The alarm was raised after activity kicked up (circle) and 24 minutes later the major goaf fall took place (square). The location of the microseismic events associated with the increased activity is also shown (inset). (Iannacchione, et al., 2005).

Rockbursts and coalbursts

Seismological precursors to rockbursts or coalbursts in underground coal mines in Australia have not been studied to date. There have been some results published for the Polish hard coal mines (Mutke et al, 2009) in which rockburst-prone zones were identified using seismic tomography. However, it is still not clear how to indicate when an impending instability would occur.

![Figure 2](image2.png)

Figure 2: The cumulative number of microseismic events vs time for a number of goafs (square symbols). Successful warnings are indicated by circles, unsuccessful warnings by triangles. Over 90% of the goafs are indicated in advance by increased microseismic activity. (Iannacchione, et al., 2005).
Rockbursts have been a significant problem for many hard rock underground mines for over 30 years [Gay and Wainwright, 1984] and there was extensive research conducted into the problem of rockburst prediction in South Africa in the 1990’s [MHSC, 2016]. While simple increases to microseismic activity rate is not a reliable precursor in this brittle environment, derived parameters like Energy Index [van Aswegen and Butler, 1993], Apparent Volume and Schmidt number [Mendecki, 1993] have been shown to provide some predictive success in the South African gold mines [van Aswegen and Mendecki, 1999]. Figure 3 shows an example where these trends in cumulative apparent volume and Energy Index were observed before a large event in a South African gold mine.

![Figure 3: A large magnitude 2.4 seismic event at TauTona gold mine in South Africa is indicated 30 hours in advance by the characteristic pattern of dropping energy index (red line) and accelerating cumulative apparent volume (blue line). Seismic events from the period of stress softening (shaded zone) can be used to identify the location of the future instability to within about 100 m. (Lynch and Mendecki, 2001).](image)

Similar precursory patterns have been observed in some Australian underground hard rock mines – for example at Beaconsfield mine in Tasmania (Hills et al, 2013) – see Figure 4.

**Figure 4:** Time history of cumulative apparent volume (blue) and energy index (red). From around 20th March 2008 the cumulative apparent volume was starting to accelerate. By 3rd April 2008 the energy index was at its lowest level since October 2007. At the time indicated by the vertical back marker, a potential instability was identified and the mine was notified. A large (magnitude 1.9) seismic event occurred in that region five days afterwards. (Hills et al, 2013).
Despite these notable successes, only a few mines around the world routinely practice “earthquake prediction”. This is due to the lack of data in most mines: many large seismic events are required to calibrate these methods, and most mines do not experience large seismic events with such regularity. Another reason is that these techniques are not 100% reliable: despite best efforts with careful seismic monitoring, many false alarms are issued and many large events are missed. This problem has not been solved yet.

For these reasons, how to reliably warn of impending rockbursts or coalbursts in underground Australia coalmines remains a difficult and open question. Addressing this question satisfactorily will require good quality microseismic data and careful research.

### SEISMIC SENSOR CONFIGURATION

Underground coalmines are typically mining a planar ore body, and naturally the tunnels and access roads all lie on this plane. A planar configuration of seismic sensors allows reliable in-plane seismic event locations but very unreliable location in the direction perpendicular to the plane. This is a problem since knowledge of whether the seismic event took place in the roof or floor strata is important for interpretation.

To solve this, geophones are usually installed into 50 m boreholes drilled upwards and downwards from the main and tail gates. This aspect ratio – 100 m of vertical separation compared with 200-300m of horizontal separation – allows reliable 3D location of seismic sources with an adequate seismic velocity model. The geophones are permanently grouted into the long boreholes for the best quality seismograms.

The expected 3D location accuracy obtained from any particular configuration of seismic sensors can be modelled if suitable assumptions are made about seismic velocity model uncertainty, seismic sensor position uncertainty and body wave arrival time uncertainties [Mendecki, 1997]. Table 2 lists the assumptions used in modelling an example 4-geophone array in the long borehole configuration just described — Figure 5 contains the results.

**Table 2: The parameters used in modelling the expected 3D location accuracy for particular sensor arrays.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Homogeneous P-wave velocity</td>
<td>4300 m/s ± 5%</td>
</tr>
<tr>
<td>Homogeneous S-wave velocity</td>
<td>2500 m/s ± 5%</td>
</tr>
<tr>
<td>Sensor position uncertainty</td>
<td>1 m</td>
</tr>
<tr>
<td>P- and S-wave arrival time errors</td>
<td>0.001 s</td>
</tr>
</tbody>
</table>

![Figure 5L: A typical long-borehole seismic sensor array for monitoring a coal longwall (left, with geophones as green triangles) and the expected 3D location accuracy for this array on the plane of the coal seam, under the assumptions given in Table 2. Such an array would be expected to provide a 3D location error of around 12 m or better.](image)
While this standard arrangement provides good coverage, it relies on long borehole and permanently installed geophones. Given the speed of mining, this is a relatively expensive solution. A more cost-effective arrangement would be to use removable sensors. These can range from sensors bolted to the sidewall (which do not provide very good data) to cementitious “swallow’s nests” (average data quality) to spring-loaded short-borehole sondes (best data quality). Figure 6 contains an image of the spring-loaded short-borehole sonde along with a comparison of data recorded by this removable geophone against data recorded by a permanently grouted geophone at the same position.

The use of removable geophones would result in a planar sensor configuration, leading to large out-of-plane location errors. To circumvent this problem, a surface seismic station can sometimes be used. When temporarily installed in a field 300-800 m above the mining, a surface geophone provides the necessary constraints to Z-coordinate error, resulting in satisfactory 3D location accuracy. Figure 7 presents the expected array performance for such a configuration.

Figure 6: A section through a removable spring-loaded geophone (above, with three orthogonal geophone elements in copper colour) and a graph showing the ratio of spectral response between the removable and permanent geophones at the same position. The shallow (1 m) borehole removable sonde gives a clean response for frequencies below 750Hz, making it suitable for use in coal mine microseismic monitoring systems.
CONCLUSIONS

The objectives of seismic monitoring in underground coal mines typically include rescue, prevention, control, warnings and back-analysis. The seismic monitoring array requirements for rescue, prevention and back-analysis are fairly loose, with geophones spaced every 1-2 km or so. However, more careful monitoring of the rock mass response to coal seam mining—the objectives of control and warning—require geophone spacing of a few hundred meters and so sensors are installed in the main and tail gates either side of the panel being mined.

The objectives of warnings are of particular interest. While warnings of impending large goaf falls has been shown to be feasible at Moonee colliery, warnings of rock- or coal-bursts is much more challenging. Goaf warnings are based on simple microseismic event activity, but this is not good enough for warnings of rock- or coal-bursts. In hard rock mines, there has been some success in warning of impending large seismic events ("rockbursts" when these are located close to excavations) based on analysis of energy index and apparent volume, which are derived from seismic event source parameters. However, before this can be applied to underground coal mines, there needs to be more quality seismic data collected and case studies compiled.

Removable borehole geophones produce good quality signals at frequencies up to at least 750 Hz and so are recommended for monitoring of individual panels. The high speed of mining means that monitoring of such panels would only be for a few months, and so it is relatively expensive to use permanently installed borehole geophones. The planar nature of the resulting sensor array results in very unreliable vertical locations of seismic events, but this can be fixed by a surface seismic station where possible.

REFERENCES


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EMERGING TRENDS IN INJURIES IN THE UNDERGROUND COAL SECTOR: AN ANALYSIS OF QUEENSLAND DATA FROM 2006-2017

Nikky LaBranche

ABSTRACT: This paper analyses the Queensland Mines Inspectorate's (QMI) Lost Time Injury (LTI) historical data set for underground coalmines. LTI data is reported to the QMI by mine operators on Form 5A. Data discussed includes: injury, body location, worksite location, occurrence class, mechanism of injury, and major equipment involved. Results are presented along with a discussion of contributing factors. Currently published analysis of these injuries only includes the number of injuries of each type. This analysis expands that information to include the severity of the injury (number of days lost and/or days on alternative duties) and the average days away for each type of incident. This analysis also includes cross-references of different data types such as injury types by body location. The analysis includes severity measures for different types of injuries related to the factors collected and drills down deeper into the data than is currently available in existing reports. The value in LTI data comes from analysing the types of incidents occurring and using that information to implement controls that prevents recurrence. This information can inform the Coal Mine Worker (CMW) of additional risks they may not have been aware of, previously the mining companies where controls have failed and the inspectorate where systemic issues are arising.

INTRODUCTION

This paper analyses the LTIs reported to the QMI for underground coal mines over the past 11 years. An LTI is described as any injury which requires a worker to either miss one or more days of work, or to not be able to perform the normal duties of their job, referred to as alternative duties. This paper looks at those incidents occurring between 1 July 2006 and 30 June 2017, covering Australian financial years FY07 to FY17 (Department of Natural Resources and Mines (DNRM) 2017a and 2017b). There were 954 LTIs reported over this period with an average of 52 days away. There were 28,162 days lost and 21,830 days on alternative duties, totaling 49,992 days away. The term ‘days away’ refers to the combination of days lost and days on alternative duties, and is used in the averages. It was identified that there were 53 lost time injuries recorded with zero days lost time or an alternative duties. This makes the numbers previously listed an underestimation of the true number of days away.

While mines and mining companies collect their own data, it is also important for this data to be collected centrally and analysed in aggregate. While one mine might have only one reported incident involving a certain piece of equipment other mines might also be experiencing the same issues in isolation. Having this analysis identifies areas of concern, which in turn identifies areas where further controls are needed. For example, the mining contractor Redpath Australia Pty Limited identified a number of LTIs due to catching hands in equipment doors and implemented the SAFE STOP Anti Door Jam Unit that recently won the Innovation Award at the Queensland Mining Industry Health and Safety conference in 2017 (Graeme, 2017). This is an example of using LTI data to identify and implement an engineering control that eliminates the opportunity for injuries of this nature to occur in the future. LTI data is most appropriately used for acute and traumatic injuries, but does not capture occupational health harms and long term exposures adequately.

METHOD

The data for this project was provided by the Queensland Department of Natural Resources and Mines DNRM) from the information collected on Form 5A. Form 5A collects a range of data including time and date, a written description of the incident, shift roster patterns, and demographics of the injured party. This data is entered at the mine site by the mining

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company. Each person entering the data for the incident is asked, as part of the submission, to code fields for the injury type, body location, worksite location, occurrence class, mechanism of injury and major equipment involved.

In order to ensure a more consistent comparison for this paper, the incident descriptions and associated coding were reviewed by the author. When it was clear that a different category in the field was a more suitable descriptor, the incident was recoded. If there was insufficient clarity in the description to identify a coding, the original data entered in the database was maintained. New categories in fields were added as appropriate when a number of similar incidents showed up, for instance categories for drilling/bolting activities and installing vent control devices were added under occurrence class. This paper analyses the number of injuries and the average and total days away for each category and presents a graph of the summarised categories for total days away.

**ANALYSIS**

Upon review, inconsistencies have been noticed in how different individuals code the same incident. This was especially evident when it was found that several incidents had been entered in the database more than once, and were coded differently. There were 24 repeats identified. The department is currently undertaking a project on LTI and HPI data collection to add clarity to the data and available options within the fields. Better coding options will allow for more accurate representations of the nature and mechanism of injuries occurring which will in turn allow for better identification of injury prevention controls.

This analysis includes an examination of injury severity as well as a count of injuries. Simply having the count of incidents does not provide an accurate quantification of the safety and health impact to workers and the industry. The average and total number of days away for each field are also considered as part of this analysis. Broadbent (2017) discusses this point in the following terms, "Guess what LTIFR strategically measures. The "small stuff". It does not discriminate between the infected finger and the amputated wrist. If you have six infected fingers and one amputated wrist you quite realistically will get a worse LTIFR result in the workplace with the infected fingers. People want to know about the amputated wrists."

**BODY LOCATION**

Of the 954 injuries, the most frequent contributor to body location is hand/finger/thumb with 195 injuries accounting for 20% of the injuries sustained. Knees and back - upper/lower each accounted for 14% of the injuries with knees slightly more frequent than backs at 137 versus 135 injuries. By contrast, knees were the third most frequently injured location in the underground metalliferous sector data behind backs and hands (LaBranche, 2017). There were 90 shoulder injuries accounting for 9% of the injuries. The 53 foot/toe injuries accounted for 6% overall. Ankle and neck injuries each accounted for 5% at 48 and 44 injuries respectively. Lower leg accounted for 35 injuries (4%) while the abdomen/pelvic region injuries accounted for 3% of the injuries. The top three categories, hands, knees and backs account for 49% of the injuries and the top five make up 64% of the injuries.

*Hand/finger/thumb* injuries with the largest number of incidents also accounted for the greatest number of days away 7970 (16%) consisting of 3553 days lost and 4417 days on alternative duties. *Knee* injuries accounted for the greatest number of days lost, 3937 and the second largest number of days away, 7474 (15%) with 3537 days on alternative duties. *Shoulder* accounted for 7471 days away, only three less than knees. While shoulders were only 9% of the injuries, they account for 15% of the days away. *Back – upper/lower* at 14% of the injuries, only accounted for 11% of the days away at 5613, consisting of 3353 days lost and 2260 days on alternative duties. *Foot/toe* injuries accounted for 1603 days lost and 968 days away (5%). *Lower leg* injuries accounted for 1504 days lost and 845 days on alternative duties while *wrist* injuries accounted for 1023 days lost and 1046 days on alternative duties (4%).

There was one *neck and trunk* injury where a CMW was struck in the back of the head and right shoulder by a rock while loading chock legs on the Armoured Flexible Conveyor (AFC)
chain which accounted for 321 days, the highest average in the body location field. There were four pelvis injuries that averaged 200 days away. There was one injury to each of the upper and lower limbs, where a CMW strained his upper arm and thigh trying to brace himself for a fall, which accounted for 159 days away. There were eight upper arm injuries averaging 127 days away and 22 wrist injuries averaging 94 days away. The 195 hand/finger/thumb injuries average 41 days away while the knee injuries average 55 days away. The back – upper/lower injuries average 42 days away while shoulder injuries are almost double that at 83 days away per injury.

A summary of days away by body location is presented in Figure 1. Many of the categories were combined for ease of viewing in the figure and combined category titles are capitalised. The figure shows that over the 11 year period Back injuries have had the largest increase in number of days away, even while having the largest decrease in the number of injuries. Over the last five years (FY13-17) there were 3504 days away from Back injuries as compared to the previous five years (FY08-12) with 1551 days away. The number of days away due to Arm injuries has also increased over time.

Figure 1: Days away by body location group

INJURY TYPE

The type of injury was analysed for each LTI. Categories were added for Bite- animal or insect, displacement of disc, infection, loss of consciousness, splinter, repetitive strain injury and whiplash. Some categories were renamed to better facilitate their use, for instance ischaemic heart disease was never used in the data set, so it was relabelled as heart attack, the one instance in the data set having been submitted as other disease. There were 127 other and unspecified injuries and 15 other disease injuries that were reclassified appropriately.

For LTIs by type of injury, sprain/strain was the largest contributor with 445 injuries accounting for 47% of the total injuries. The next largest category was fracture (not of the vertebral column) with 160 injuries (17%), followed by contusion with intact skin surface/crush with 131 injuries (14%). There were 67 open wound injuries (7%) and 24 traumatic amputation injuries (3%). There were 18 dislocation injuries accounting for 2% of the injuries.

The most significant contributor to the total number of days away was sprain/strain injuries with 12,137 day lost and 11,585 days on alternative duties (47%). Fracture (not of the vertebral column) account for the second highest number, with 6511 days lost and 3865 days on alternative duties (21%). These were followed by contusion with intact skin surface/crush at 3002 days lost and 1814 days on alternative duties (10%) and open wound injuries at 1828
days lost and 1990 days on alternative duties (8%). Traumatic amputations accounted for 519 days lost and 673 days on alternative duties (2%) while dislocations accounted for 889 days lost and 632 days on alternative duties (3%).

There were seven fractures of the vertebral column averaging 107 days away ranging from a reported three days away to 292 days away. There were five poisoning/toxic effect injuries averaging 98 days, three of these being a single day, with the largest amount of time off being 450 days for the Lectra Clean (1-Bromopropane electric motor and equipment cleaner) chemical exposure, which resulted in Sintars research to recreate the potential exposures (Djukic 2017). The 18 dislocations averaged 85 days away, while the 11 mental disorders averaged 68 days away. Fracture (not of the vertebral column) injuries averaged 65 days away while sprain/strain injuries averaged 53 days away.

Figure 2 shows the days away for the top six injury types by financial year. The injury types are too dissimilar to meaningfully combine, so this figure shows only the top contributors and does not account for all the days away for the time period. Fracture (not of the vertebral column) and Contusion with intact skin surface/crush have both had an increase in the number of incidents and the total days away for those incidents over the 11 year period. Sprain/strain has had the most dramatic decrease in the number of incident and number of days away.

Figure 2: Days away by injury type

OCCURRENCE CLASS

The LTIs were analysed by occurrence class. Several new categories were added as necessary to provide better data clarity based on the types of incident in the dataset. Categories were added for drilling/bolting and installing vent control devices. A category for no specific incident was added to cater for the repetitive motion injuries that occur over long periods of time. A category for equipment ingress/egress was added for incident involving getting on and off equipment. This includes vehicles like LHDs and personnel transporters. The injuries while getting on/off the longwall beam stage loader (BSL) and district control breaker (DCB) are included as discrete pieces of equipment in equipment ingress/egress, but slip/trips falls on the longwall face itself are categorised as other equipment access e.g. moving about. A category for proximity to operating equipment was added to cover instances where the CMW was operating the equipment as intended or near the equipment during operation, for instance a rock striking a CMW while operating the shearer or standing near a bolter when struck. The working on equipment category was used for repair and maintenance activities. A category for transporting with lifting aids was added, this includes cranes, jacks and chainblock.

The largest category in occurrence class was moving on foot with 159 injuries (17%). There were 122 injuries (13%) working on equipment. Transporting manually i.e. carrying, dragging
and drilling/bolting each had 106 injuries (11%). There were 84 other manual handling injuries (9%) and 57 equipment ingress/egress injuries (6%). There were 53 injuries (6%) related to proximity to operating equipment. There were 51 injuries (5%) travelling in equipment/vehicle, 24 of which were LHDs. Operation of non powered hand tools accounted for 37 injuries (4%). Others loading/unloading from vehicles each had 28 associated injuries (3%).

The largest number of injuries (159), moving on foot also account for the most days away with 5013 days lost and 4038 days on alternative duties (18%). Transporting manually i.e. carrying, dragging was the next most significant contributor to days away with 2669 days lost and 3005 days on alternative duties totaling 5674 days away (11%). Working on equipment injuries accounted for more days lost (3765) than transporting manually i.e. carrying, dragging, but significantly less days on alternative duties (1778), totaling 5543 days away (11%). Drilling/bolting accounts for 2113 days lost and 2116 days on alternative duties (9%). Other manual handling accounts for 2069 days lost and 1875 days on alternative duties (8%), while travelling in equipment/vehicle accounts for 2294 days lost and 1174 days on alternative duties (7%). Proximity to operating equipment accounts for 2332 days lost and 1059 days on alternative duties (7%).

There was one occurrence of ascending – ground/floor not involved that had the highest average number of days away at 95 days where a CMW was climbing up onto a chock using the inter-chock hoses as a step. The next highest average was 84 days away for the 26 other equipment access e.g. moving about injuries. The 37 instances of operation of non powered hand tools averaged 75 days away while the five welding related injuries averaged 72 days away. Incidents related to traveling in equipment/vehicle averaged 68 days away. The 159 moving on foot injuries averaged 57 days away while the 122 working on equipment injuries averaged 45 days away. There was an average of 54 days away for the 106 transporting manually i.e. carrying, dragging injuries. For the 106 drilling/bolting related injuries there was an average of 40 days away. The 84 other manual handling injuries averaged 47 days away while the 57 equipment ingress/egress injuries averaged 48 days away.

A summary of the types of occurrence classes that contributed to days away is shown in Figure 3. Many occurrence classes have been combined to make the categories easier to view. The number of drilling and bolting related injuries have increased over the time period, while there was also a slight increase in equipment access. The number of Travelling in equipment related injuries has remained steady over time. The largest decrease over time has been to manual handling injuries, followed by Operating tools/equipment and Moving on foot with other being the smallest decrease.

![Figure 3: Days away by occurrence class group](image-url)
MECHANISM OF INJURY

The LTIs were also analysed by mechanism of injury. When recoding the data, any incident of a rock hitting a person was categorised as fall/slide/cave-in of material, and taken out of being hit by falling object if previously categorised that way, raising the number of incidents from 20 to 72. New categories were added for equipment ingress/egress as a more specific type of slip/trip/fall, for infection and for no specific incident for long term injuries. There were 43 of the 44 injuries originally submitted at unspecified mechanism of injury and 25 of the 26 submitted as other and multiple mechanisms of injury that were able to be assigned to the appropriate category.

Fall/slip/trip on the same level accounts for 150 injuries (16%) while muscular stress - lift/lower/carry object accounted for 134 injuries (14%). There were 104 instances of being trapped between stationary and moving object (11%), while there were 80 instances of being hit by a moving object (8%). Fall/slide/cave-in of material-underground is now at 72 injuries (8%) while there were 61 instances of being hit by a falling object (6%).

Fall/slip/trip on the same level was the largest contributor to days away with 4510 days lost and 8545 day on alternative duties (17%). Muscular stress - lift/lower/carry object accounted for 3583 days lost and 3454 days on alternative duties (14%). Being trapped between stationary and moving objects resulted in 2874 days lost and 2312 days on alternative duties (10%), while being hit by a moving object accounted for 2952 days lost and 2616 days on alternative duties (11%). All types of Fall/slip/trips accounted for 22% of days away while all Muscular stress related injuries accounts for 25% of the LTIs. Being Struck by/striking/trapped by object was the largest contributor with 32% of the days away.

The two vehicle collision incidents averaged 113 days away, one being 223 days away for a shoulder injury. The five exposure to mental stress factors injuries averaged 99 days away while the 10 single contact with chemical/substance averaged 81 days away. Other variations in pressure averaged 72 days away while being hit by moving object averaged 70 days away. Fall/slip/trip on the same level averaged 57 days away while muscular stress- lift/lower/carry object averaged 53 days away.

The frequently occurring mechanisms of injury were rolled up into their larger themes to analyse how the mechanisms have changed over time, shown in Figure 4. Over the decade there has been an increase in the number of and days away from injuries caused by Material Movement and Muscular Stress. Due to miscoding this has gone largely undetected in the data set with only 20 injuries being originally coded as fall/slide/cave-in of material - underground and 71 being identified upon review of the data. While the number of Equipment Motion and Access injuries has slightly decreased there has been a slight increase in the days away. The reverse has happened for Struck by/striking/trapped by object, where the number of injuries has increase, but the days away has decreased over time. Fall/slip/trip and Other have decreased both in number and days away.

WORKSITE LOCATION

A significant amount of recoding was done to the worksite locations. This included 70 of the 122 other underground location, 17 of the 18 unknown underground location and 47 of the 62 coalface – working unspecified being recoded to a more appropriate descriptor. Instead of using coalface 1st working and coal face – 2nd working, which submitters found confusing, the categories for development were split into coal face – CM (continuous miner), CM Section (outbye face to feeder breaker), CM flitting and CM – second working (floor brushing, pulling pillars, etc.). There were also categories created for coalface – place change mining and coalface – place change bolting. The reclassification increased the number of coalface – longwall injuries from 88 to 182 (19%) averaging 55 days away with a total of 5,745 days lost and 4,232 days on alternative duties (20%). Coal face – CM accounts for 159 injuries averaging 54 days away and including 4698 days lost and 3905 days on alternative duties (17%).
Figure 4: Days away by mechanism of injury

MAJOR EQUIPMENT INVOLVED

There were a great number of inconsistencies in the major equipment involved category, which required a significant amount of rework during recoding. Performing maintenance activities and instances where more than one piece of equipment is involved are particularly prone to errors based on the way the system is set up. For instance, when lifting double acting (DA) cylinders from the front AFC with a chainblock there was some confusion as to whether longwall chock, non-powered lifting equipment e.g. jack, chain block or other non-powered equipment/object would be the appropriate category. These were all recoded to reflect the action being taken that contributed to the injury, so in this case the non-powered lifting equipment e.g. jack, chain block was coded. For future data collection by the Department, the author recommends that any injuries sustained while working on equipment have the ability to code data on both the tool and the piece of equipment being worked on.

CM bolting accounted for 63 injuries (7%) and averaged 36 days away with 1140 days lost and 1118 days on alternative duties. When combined with drilling rig (10) and roof support (not L wall chock) (6), the number of injuries for Drilling/bolting jumps to 122, the largest of any type of equipment. If you cross reference this combined bolting group with the mechanism of injury an overwhelming 82 of these are related to being Stuck by/striking/trapped by object, while 15 are related to Material movement, 14 to Muscular stress and 11 to Fall/slip/trip. Other than the 82 Stuck by/striking/trapped by object, 55 are hand/finger/thumb injuries and 12 are foot/toe. This suggests that there is room for reviewing these instances where the same injuries are repeatedly occurring the same way for ways to improve the system.

There were 91 injuries (10%) with no equipment involved (176 before recoding), many being slips/trips/falls, which averaged 39 days away and accounted for 2037 days lost and 1489 days on alternative duties. None also includes the 10 no specific incident injuries. Separate categories for cable, pipe and hose were combined into cable/pipe/hose – not pressurised with 62 entries (6%) as these most frequently represented the same type of manual handling injury as opposed to pressurised pipe/hose/gas cylinder, with 27 injuries which together make Cable/pipe/hose. Cable/pipe/hose – not pressurised averaged 58 days away and accounted for 1140 days lost and 1118 days on alternative duties (5%). Many of these involved muscular stress while lifting/carrying or a slip/trip/fall. Longwall includes the longwall chock which accounted for 59 injuries combined with longwall armored face conveyor (6), longwall shearer (28), longwall – BSL (8), longwall – DCB (3) and longwall - other equipment (16) totaling 120 injuries, only slightly behind drilling/bolting. Longwall chock averaged 51 days...
away and accounted for 1689 days lost and 1929 days on alternative duties (6%). Most of these involved slip/trip/falls or being hit by falling rock.

Conveyor consists of belt conveyor (12 injuries) and conveyor structure was added as a category to capture the 47 injuries related to pulling structure and changing rollers. CM/shuttle car includes the 39 CM, 18 CM cable, 14 shuttle car – underground, 7 feeder breaker (coal) and 2 trailing cable to machine (shuttle car) injuries.

Figure 5 shows that over the period Drilling/bolting injuries had the largest increase in the number of injuries over time and a slight increase in the number of days away. Longwall related injuries had a slight increase in the number of injuries and the largest increase in the number of days away. CM/shuttle car related injuries had the largest decrease in the number of incidents and in the total days away.

CONCLUSIONS

For the 954 injuries occurring between 1 July 2006 and 30 June 2017 in underground coalmines there were 28,162 days lost and 21,830 days on alternative duties, totaling 49,992 days away. Overall these incidents averaged 52 days away. The overall LTI frequency rate has plateaued since FY12 while the contractor LTIFR has increased in the past 2 years. The days away per million man-hours has decreased over the period.

The recoding and analysis performed so far has served to appropriately categorise the data to identify the trends and area of interest. There is still a significant amount of work that can be done in further investigation of the injuries in this data set to put together prevention strategies to guard against future injuries of the same nature in the industry. The project the Department has undertaken to revamp the Form 5A seeks to rectify some of the current mistakes made in coding and improve information collation for analysis. For example one common mistake found in coding was rock fall injuries being codes as being hit by falling object instead of the fall/slide/cave-in of material – underground. As a result this has emerged as a much larger risk than previously thought with the number of incidents identified rising from 20 to 72.

Hands are the most likely body location to be injured and care should be taken in these areas to minimise future injuries, especially around drilling and bolting activities. Of the 122 drilling/bolting accidents 58 were hand injuries and the number of traumatic amputations related to these have increased in recent years. The number of fractures has increased with record highs in FY17 of 26 fractures and 12 of those being hands, the next highest annual total being FY14 with 20 and a tie for 9 hand fractures in FY08 and FY12. The most likely injury mechanism for drilling/bolting is being trapped between stationary and moving object (37) followed by being hit by falling object (21) which is usually the drill steel or bolt falling on
them. There were also 16 fall/slide/cave-in of material – underground incidents associated with drilling/bolting. There is a rising trend in the number of fractures sustained that should be investigated further.

Knees are the second most likely body location to be injured, accounting for 14% of the injuries as opposed to 8% of the underground metalliferous sector injuries. Of the 137 knee injuries Fall/slips/trips account of 74 of them with 49 due to muscular stress. The most common occurrence class is moving on foot with 168 injuries, 67 of those being knee injuries.

ACKNOWLEDGEMENTS

The author would like to thank the Department of Natural Resources and Mines for providing the data and Russell Albury, the Chief Inspector of Coal Mines for his advice and support.

REFERENCES

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INTERPRETATION OF CLEAT FROM IMAGE LOGS

David Titheridge

ABSTRACT: Resistivity and acoustic scanner image logs, in both the Coal Seam Gas (CSG) and coal-mining industries, are the predominant means of determining azimuths of joints/cleat in coal. This paper indicates the need for care if interpreting cleat azimuths from image logs.

The value of cleat and joint azimuth information, and horizontal stress azimuths, is in planning the optimal orientation of deviated in-seam (lateral) production wells (CSG) and in-seam gas drainage holes. Image logs of the bore wall often exhibit large fractures (joints) that intersect the entirety of the bore wall. They are visible as sinusoidal traces. Those fractures that have low height (“cleat”) and intersect one or both sides of the bore wall are represented by vertical to sub-vertical linear traces. Acoustic image logs often only record joints.

An image log of a cleat lineation records the bore-wall intersection azimuth (BIAZ), that is an apparent azimuth, as well as the apparent dip (or plunge) of the lineation. The best way to determine true cleat azimuths from lineations on an image log is from a statistical weighted mean of numerous apparent azimuths (BIAZ).

INTRODUCTION

The CSG industry routinely uses image logs to determine the azimuths of joints and cleats in coal seams. Titheridge (2014) developed an alternative empirical method of determining cleat azimuths using the presence of coring induced tensile fractures (CITF), mainly petal fractures, and breakout on an image log. In those CSG wells. Where there is no core recovered, similar fractures in the bore wall are referred to as drilling induced tensile fractures (DITF). The empirical CITF method has provided an opportunity to compare empirical cleat azimuth results from CITF and interpretation of cleat azimuth from image logs. In several wells examined in 2008, it was found that cleat azimuth results determined by the CITF method differed from image log interpretations by a service provider, by 90°. Recent observations of similar differences in an unpublished company report (2014), has prompted a review of the geometric principles of image log interpretation.

BACKGROUND

The aim of CSG exploration is to obtain information that will allow planning of CSG production wells, as well as assess lateral variation in production. Successful CSG production is substantially dependant on gas content, gas saturation and permeability. This paper focusses on attributes affecting initial permeability (cf. the changes in permeability that occur during production of CSG or mine gas drainage). Initial permeability is inversely related to the magnitude of the normal stress component of the principal horizontal stresses acting on cleat (Titheridge, 2014). It is assumed initial permeability can be expected to provide some indication of future production.

The primary determinants of permeability are the interconnectivity of joints and cleat, and the effective normal stress magnitude acting on cleat (Table 1). The latter determines cleat aperture. In most instances the major horizontal stress azimuth (S_H) is (near) parallel to cleat but in some instances may range from oblique to perpendicular to cleat and joints. Hence a knowledge of the azimuths of cleat and joints in coal, as well as in situ stress azimuths and magnitude is fundamental to CSG production (Bell, 2006, Gray and See, 2007, Titheridge, 2014 and Mukherjee et al., 2017).

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University of Wollongong, February 2018
Table 1: Factors affecting initial* permeability of coal

<table>
<thead>
<tr>
<th>Major factors</th>
<th>Other factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Cleat/joint interconnectivity</td>
<td>3. Angle between $S_h/S_i$ and face cleat</td>
</tr>
<tr>
<td>Height ↔ length ↔ spacing</td>
<td>$S_h \perp FC$</td>
</tr>
<tr>
<td>Number of cleat generations</td>
<td>$90^\circ \pm \angle S_h \perp FC \geq 0^\circ$</td>
</tr>
<tr>
<td>(i). Primary face cleat only</td>
<td>high ↔ intermediate ↔ low</td>
</tr>
<tr>
<td>(ii). Face and butt cleat</td>
<td>(determines magnitude of normal stress acting on cleat)</td>
</tr>
<tr>
<td>(iii). Multiple cleat azimuths</td>
<td></td>
</tr>
<tr>
<td>2. Fault stress regime and principal stress magnitudes</td>
<td>4. Mineralisation of cleat</td>
</tr>
<tr>
<td>4. Mineralisation of cleat</td>
<td></td>
</tr>
<tr>
<td>5. High CO2 (%) and content; coal mining and CMM</td>
<td></td>
</tr>
<tr>
<td>- causes swelling</td>
<td></td>
</tr>
</tbody>
</table>

*before matrix shrinkage associated with production/desorption

Image logs, for determination of joint and cleat azimuths, and breakout, are of three types, namely resistivity, acoustic and optical. With regard to vertical drill holes, each type has advantages (detail of structures) and disadvantages (cost, limitations in application). Logs used in coalmine exploration are mainly acoustic. In this paper, the term “joint” refers to a planar structure that intersects the entirety of the bore wall, and is represented by a sinusoidal trace on an image log. The term “cleat” refers to smaller scale features that intersect one or both sides of the bore wall, and appear as lineations on an image log (Figures 1 and 2; cf. definitions of Laubach et al., 1998, and Dawson and Esterle, 2010).

Figure 1: Representations of a planar joint

a) In a borewall “cylinder”. b) Sinusoidal trace of planar joint on a rolled-out cylinder wall. Strike 90/270. Dip 70° to 180. c) Image log of an interpreted joint and smaller scale cleats.

AZIMUTHS OF JOINT PLANE AND CLEAT LINEATIONS FROM IMAGE LOGS AND CORE

Joint plane azimuths from measurements of the properties of a sinusoidal trace

Joints with a dip of less than 90° intersect the entirety of a bore wall if they are of sufficient height. They are represented by a sinusoidal trace on an image log. The azimuth of the minima of the sinusoidal trace is the direction of dip. This can be represented by a lineation on a stereonet or polar net (with a plunge and trend). The strike of the joint is orthogonal to the dip azimuth (Figure 1). Determination of joint azimuths comprises individual measurements of single fractures. Most fractures recorded by image logs from vertical drill-holes have a dip of 80-90 degrees. In this and subsequent sections, E-W strikes with a dip of 70 degrees for both planes and lineations are used for the purposes of explanation of principle, and discernible separation of very similar apparent dips on a stereo - or polar net.
Figure 2: Images of fractures (cleat) on a scanner image

- **a)** Diagram of two cleats, one enters the bore wall on one side, the other passes through the entirety of the bore wall. Both have limited height and do not intersect the bore wall in the up- or down-dip part of the hole.
- **b)** Two paired cleats, same depth with opposite dip. Strike = \((\text{BIAZ}_1 + \text{BIAZ}_2)/2\) ± 90. In addition to direct calculation these lineations can be counted in a statistical population.
- **c)** Unmatched single cleats -- can only be analysed as part of a statistical population.

**CLEAT AZIMUTH FROM BOREHOLE INTERSECTION AZIMUTHS (BIAZ): METHOD 1.**

Smaller scale cleat intersecting the bore wall on one or two sides of the bore wall appear as lineations on an image log (Figures 1c, 2b and c). These features have limited height and as a result do not intersect the bore wall in the up-dip or down-dip part of the hole.

In most situations, cleat on an image log records an apparent azimuth (BIAZ) with an apparent dip or plunge (Figure 2). The true azimuth is associated with the largest number of cleats in a designated azimuth class (generally 10 degrees) intersecting the bore wall (Figure 3). This class is also associated with the highest number of apparent dips.

![Figure 3: Rose diagram of 15 degree increments that results from a plot of bore wall intersection azimuths (BIAZ) of one uniformly oriented E-W cleat set that dips 70° South](image)

- **a)** Relative percentages within each 15° interval (3-26%).
- **b)** The E-W trending green cleat (highlighted in light yellow) has an apparent dip (AD/plunge) of 63 degrees, it intersects the bore wall at 045 (BIAZ). The strike of the section tangential to the bore wall (STS), is 045+90=135°/315°. The STS of the cleat that passes through the centre of the hole is 0°/180°.

The only instance where the BIAZ is the true strike is where the cleat (or its extrapolated path) passes through the centre of the bore-hole; this BIAZ is also coincident with the true maximum dip (Figure 3).

As most of the BIAZ of cleat are apparent azimuths, then one or several individual BIAZ are not sufficient to determine cleat azimuth. As an approximate rule of thumb, at least 20 BIAZ within a 50 degree range are required to determine the mean cleat azimuth. In most cases, cleat BIAZ are conveniently presented as a rose diagram or a histogram (Figure 4). A weighted mean of the numbers of the BIAZ in the vicinity of maxima will provide a close measure of the true azimuth.
Figure 4: Method 1: Cleat azimuths based on number of BIAZ in 10° classes for DH “A”
    a) Polar plot of BIAZ and apparent dips (10° classes). Highest apparent dips are closest to the graph perimeter.
    b) Rose diagram (number of BIAZ) in diametrically opposite 10 degree classes.
    c) Histogram with same data.

TRANSFER OF IMAGE LOG BIAZ DATA TO A STEREONET TO DETERMINE STRIKE

Bore walls are circular sections and therefore the strike of the section on a bore wall is tangential to the bore wall at the point where a cleat intersects the bore wall. Where, for example, the BIAZ of a cleat is 045, the strike of the (tangential) section (STS or trend) is 135. The plane containing two or more apparent dips on a stereo-net defines the true dip and strike. To obtain strike and true dip from image log data, it is necessary to rotate the line containing the BIAZ and apparent dip by 90 degrees so that the STS (trend of lineation) is correctly represented on the stereo-net (Figures 5 and 6). If this is done for two or more lineations intersecting the bore wall, at different distances from the centre of the bore, and with the same strike, the plunges of the lineations will define a great circle on the stereo-net.

The process illustrated in Figure 5 demonstrates that a wide range of BIAZ can be a result of many cleats with the same azimuth intersecting the bore wall at different distances from the bore wall centre.

Figure 5: Rotations of apparent dip and strike of lineation to determine true dip
    a) Five cleats with E-W strike and dip of 70°S, and labelled 1 to 5. Cleat 1 passes through the centre of the core with an azimuth of 090. b) Five cleats with BIAZ and plunge (diamonds). c) Rotation of cleats by 90° to STS orientation. d) Rotation of stereo-net so that all the apparent dips fit a great circle on the stereo-net. Apparent dips (plunges) of cleats 1 to 5, define the E-W plane and dip 70°S. Any two apparent dips will define true dip and strike.

ALTERNATIVE WAY OF DETERMINING THE AZIMUTH OF CLEAT LINEATIONS: METHOD 2.

Some unpublished interpreted image log diagrams that have been generated by software and provided to the CBM industry, indicate a different method has been used to determine the azimuths of cleat represented by lineations on an image log. The method is not described in the reporting of interpretation but it has been possible to deduce the underlying rationale and process from the fortuitous inclusion of a polar plot with a single joint and a single cleat (subsequent section).

Figure 6 compares the representation of a dipping plane from an image log and the determination of cleat azimuth from image log lineations via Methods 1 and 2. The results have an exact difference of 90°. The determination of cleat azimuth via Method 2 is similar to the determination of the strike of a plane, whereby the azimuth of a plane is orthogonal to the direction of dip (lineation trend).
Figure 6: Comparison of Method 1 and Method 2

a) “Image log” E-W joint (pink sinusoid), dip 70°S. Red cleat (090) passes through centre of bore hole whereas cleats with BIAZ of 060 (green), and 105 (brown) do not. b) Joint (pink) with E-W strike dips to south (trend and plunge indicated by pink diamond). E-W cleat lineation with plunge of 70 to E (red diamond). c) Rotation of cleat to N-S (STS) to represent actual plunge. Note the coincidence of joint and cleat dip lineation. d) Method 1. Cleat azimuth determination involves statistical analysis of BIAZ data (060, 090, 105). The cleat with an azimuth of 090 passes through the centre of the core and therefore it must have the same strike as the plane that contains it (pink sinusoidal trace of “a”). e) Method 2. The azimuth of the red cleat is taken to be orthogonal to cleat lineation that plunges 70° to 090 (double arrow). f) The azimuths of cleats with BIAZ of 060 and 105 are also rotated 90°.

DETERMINATION OF CLEAT AND JOINT AZIMUTHS FROM CORE

Examples of petal fractures and their relationship to horizontal stress in core and the bore wall are illustrated in Figure 7. Determination of cleat/joint azimuths from core involves measurement of the angle between a cleat and the apex of a coring induced petal fracture on the bore wall.

The basis of cleat azimuth determination is that the strike of petal fractures has the same azimuth as $S_H$, and the apex of the trace of a petal fracture has the same azimuth as breakout on an image log. Cleat azimuth can be determined by measuring the angle between cleat/joints and the apex of a petal fracture (formed at the intersection of the petal fracture and the core circumference). Details of the method of calculation of cleat azimuth using petal fractures are illustrated in Titheridge, 2014.

Figure 7: Petal fractures and their relationship to horizontal stress and breakout azimuths

a) and b) Top and side view of petal fractures in core  c) Fracture/stress relationships in core and bore wall.
COMPARISON OF JOINT/CLEAT AZIMUTHS FROM IMAGE LOGS AND THE CITF METHOD

In 2007/08 image logs were used to obtain cleat azimuths from four wells (DH’s “A”, “B”, “C” and “D”) from a CBM lease. In addition, cleat azimuths were obtained using the presence of coring induced petal fractures (method outlined above, Titheridge, 2014). The results are presented in Figure 8, with the results via the CITF method overlain on the image log results.

The numbers of azimuth determinations via the CITF method is low, as at least half the coal core had been removed for gas desorption and destructive testing prior to applying the CITF method. It is estimated that as many as three to four times as many cleat azimuth determinations could have been obtained had the measurements been made whilst the coal was still in the splits, or if the core had been marked to preserve orientations of adjacent pieces of core.

Regardless of the handicap outlined above, a comparison of the measurements of the CITF method and the image log interpretations of a service provider was made (Figure 8). In Holes “A” and “B”, the major cleat azimuths from the CITF method was approximately orthogonal to the image log interpretation by the service provider (Figure 8). The differences prompted enquiry with the service provider concerning their image log interpretation. In the final hole to be drilled in the series, Hole “D”, the results from the image log interpretation of the service provide and the CITF were nearly identical. This prompted enquiry as to how the differences arose in Holes “A” and “B”.

A COMPARISON OF METHODS TO OBTAIN AZIMUTHS FROM CLEAT LINEATIONS

The cleat azimuths determined by Method 2 for Holes “A” and “B” indicate the fractures are perpendicular to $S_H$ (Figure 8) and suggest an unfavourable stress/fracture azimuth scenario for permeability (Table 1). It is noteworthy that if the image log azimuths for cleat lineations (BIAZ) for Holes “A”, and “B” are rotated by 90°, there is a reasonable fit between the azimuths of both the Method 1 image log interpretation and the CITF method (Figure 9a), and a more favourable stress/fracture azimuth scenario for permeability.

Conversely if the same rotation of cleat lineation BIAZ (Method 2) is applied to Hole “D” (Method 1 and CITF method), then the cleat will be perpendicular to the joint azimuths, determined from a sinusoidal trace (middle rose diagram Figure 9b). Observations of core do not support this, and it (generally) does not make geological sense for the larger scale joints to be nearly orthogonal to the cleat represented on the image logs by lineations. This is supported by core observations where there is clearly only one cleat and one joint azimuth that are very similar.

Figure 8: Cleat azimuth determined by image log Methods 1 and 2, and the CITF method.
REPRESENTATION OF STRUCTURES DATA WITH TADPOLE PLOTS.

Tadpole plots graphically record both dip and azimuth of structures (Table 2) and their depths.

Table 2: Information from dip log tadpoles

<table>
<thead>
<tr>
<th>Intersection of fracture with bore wall</th>
<th>Fracture/ Form</th>
<th>Tadpole tail information</th>
<th>Numerical format of tadpole information</th>
<th>Obtaining Strike AZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Entirely</td>
<td>Joint/Sinusoid</td>
<td>Dip direction(trend)</td>
<td>True dip/true dip AZ</td>
<td>Orthogonal to true dip</td>
</tr>
<tr>
<td>Side (Method 1)</td>
<td></td>
<td>Orthogonal to strike</td>
<td></td>
<td>Mean of BIAZ</td>
</tr>
<tr>
<td>Side (Method 2)</td>
<td>Cleat/Lineation</td>
<td>Plunge trend;</td>
<td>Plunge/BIAZ</td>
<td>Mean of orthogonal to BIAZ</td>
</tr>
</tbody>
</table>

In addition to the graphic tadpole plots, some logs also record the same information in numerical form. The y-axis of the tadpole plots is depth, the x-axis records dip of planar structures that intersect the entirety of the bore wall with a sinusoidal trace, as well as the apparent dip of cleat lineations. The tail of the tadpole is effectively a “z” axis that records true dip azimuth of planar structures and the BIAZ of lineations (Table 2).

AZIMUTHS FROM JOINTS AND CLEAT IN CLOSE PROXIMITY: METHODS 1 AND 2

In Hole “D” numerous inclined joints (top inset LHS Figure 10), are present from 789 to 799m. The height of the planar joint indicated by the pink sinusoidal trace is about 80cm (indicated by blocky pink arrows). This joint has a dip of 75°, a dip direction of 035° and strike of 125° (see bottom expanded inset). A cleat lineation “A”, with a BIAZ of 117° and plunge of 73° is in close proximity to another lineation “B”. The sinusoidal trace is based on fitting a curve to lineations “B” and “C”. These data are plotted on a series of polar plots (a to c) in Figure 10 (RHS).
Figure 10: Application of Methods 1 and 2 applied to drillholes “D” and “E”

The RHS of Figure 10 illustrates a polar net analysis to test the results of applying Method 1 vs Method 2, to the joint that is in close proximity to cleat “A”. In Figure 10a (Method 1A), the procedure consists of plotting the strike of the joint, and the BIAZ (trend) of cleat “A” and its plunge. The difference between the azimuth of the plane and the trend of the lineation is 8 degrees. In Figure 10b (Method 1B), the lineation is rotated 90° to the strike of the tangential section of the lineation of 027. The rotated lineation is the true lineation trend. This process of rotation makes the dip and trend lineation of the plane and the cleat lineation almost identical (refer back to Figure 6c). The orthogonal to the restored lineation is 117 (= BIAZ), and produces the same result as Method 1A.

If Method 2 is used (Figure 10c), and the interpreted strike is drawn perpendicular to the dip and trend of the lineation, then the difference between the strike of the plane and the cleat is large, when, as indicated by their close proximity and dip, they should be similar. There is also a large difference between the trend of the dip lineation of the plane, and the cleat. Whilst Methods 1A and 1B have near identical azimuth results, it is only Method 1B that has near identical results for both trend and plunge of both lineations. Method 1, along with statistical analysis, is therefore the preferred method to determine azimuth of cleats.

In the inset in the bottom right of Figure 10, some image log snapshots from Hole “E” (Bowen Basin) are presented in Figures 10 d,e,f,g and h. The observations from the image log report of Hole “E” have been fundamental to the interpretation in this document. Figure 10d illustrates a sinusoidal trace (blue green) with a cleat in close (circled white and enlarged in Figure 10e). In Hole “E” joint and cleat details (azimuth and dip/plunge) for each 2m interval were presented. In most figures it was not possible to identify individual joint and cleat details (Figure 10f) because of the abundance of data and the plotting with a scale range of 0-90°, for dips that are mostly greater than 80°.

In the example illustrated in Figure 10g, there were only two structures in a 2m summary interval. The figure shows a joint that strikes approximately N-S, with an adjacent cleat with similar dip striking nearly E-W. This interpretation of the E-W cleat emanates from Method 2. If Method 1 is applied, the interpretation of the cleat azimuth is approximately N-S (red line of Figure 10h) and similar to the strike of the joint, and consistent with Hole “D” in Figures 10 a and b.

INTERPRETATION OF THE DISPERSION OF CLEAT AZIMUTHS BASED ON LINEATIONS.

Figure 3 illustrates a dispersion model of BIAZ lineations associated with a known single E-W azimuth. Figure 11 (below) shows examples with considerable variation in the number of lineations (one to hundreds), and the number of azimuths (one to six). In Figure 11, lineations (recorded as dots on polar plots) appear to be treated as measured individual azimuths (as is
correct in the case of joints), rather than dispersion of one or several populations. This is indicated by the automated (italicized) interpretations, below each of the rose diagrams. These interpretations are debatable.

**Figure 11:** Examples of the pattern of the distribution BIAZ

*Numbers of BIAZ data range from 1 to 393. Author’s interpretations of azimuth in c) and d) indicated by red lines.*

Figure 11a is based on one lineation. With reference to Figure 3, it is estimated there is only about a 15% chance that the single lineation is within the 10 degree azimuth class that contains the (true) strike. Figure 11b is described as having a trimodal distribution. There are insufficient measurements to be certain this is correct. It is easily possible there are only one or two azimuths. If lineations on an image log are sparse, inspection of core, or the use of the CITF azimuth method, could resolve the modality and the true azimuths. Figure 11c describes the distribution as “scattered”. Three modes are cited in Figure 11c. There is probably a fourth (~ 087). Within each of the azimuth cases, the range of azimuths is probably narrow rather than scattered; the “scatter” (dispersion) is an artefact of most fractures not passing near the centre of the bore wall (see Figure 3). In Figure 11d, the “scattered” distribution is more likely to be due to the presence of four distinct azimuths, each with a narrow range; inspection of core could easily resolve this.

**CONCLUSIONS**

The intersection of cleat on one side of a bore wall produces a dipping lineation. The bore wall intersection azimuth (BIAZ) is an apparent strike and is associated with an apparent dip. The strike of the tangential section (STS) at the BIAZ contains the true trend and plunge of the lineation. This must be accommodated in plotting lineations from image data on a stereo-or polar-net.

It is suggested that the best way to determine true azimuths of cleat lineations from an image log is from a statistical weighted mean of numerous bore-hole intersection azimuths (BIAZ; Method 1). In Method 2, a 90° rotation of bore-hole intersection azimuths (BIAZ) by others appears to be a software procedure. In effect, the apparent dip of a lineation has been treated as if it were the plunge trend of the dip of a joint. Differences in interpretation of cleat azimuths of 90° between Methods 1 and 2 are likely to have implications for planning the azimuths for in-seam production wells, and ultimately gas production. To convert Method 2 to Method 1 results, there is a simple remedy – rotate the BIAZ cleat lineation azimuths (another) 90°.

In the absence of documentation of the method of cleat azimuth determination by service providers, the end users (geologists, production engineers) need to verify the method used. The value of inspecting core to supplement image logs cannot be over-emphasized. The results of application of the CITF method has prompted a closer examination of the interpretation of cleat azimuths from lineations on image logs and provided the impetus for this paper. The angle between stress azimuth, as indicated by petal fractures, and cleat azimuth, is an empirical measurement. It can be used to ground truth cleat azimuth determined from a statistical interpretation of lineations on an image log.
ACKNOWLEDGMENTS

The author is grateful for the constructive comments of Professor Joan Esterle, Malcolm Bocking and Alfred Lacazette, and Saswata Mukherjee for providing access to the contents of Figure 11.

REFERENCES

ABSTRACT: The risk of fatalities from rib failure is still prevalent throughout the coal mining industry which prompted further industry research into understanding rib deformation and rib support interaction. This paper provides the results of a rib deformation monitoring project at Moranbah North Mine as part of ACARP project C25057. Moranbah North Mine provided funding and mine site access to assist this research into the risk of rib failure within the industry. Two rib monitoring sites were installed to monitor rib and roof deformation and rib bolt loads under both development and longwall retreat stress scenarios. The monitoring highlighted the progression of rib deformation from the minor deformation and bolt loads experienced on development, through to the significant deformation observed under longwall abutment loads. The stability of the Tonstein Band (10 to 15 cm claystone/siltstone band located approximately 1 m above the floor), was highlighted as a key factor in the rib deformation both on development and retreat. The monitoring provided observations of the progression of deformation and highlighted a step change in rib stability. The rib deformation stepped from near rib deformation within the bolted zone to a significant increase in depth of softening under longwall abutment loading. It was inferred from the monitoring data that shear failure along the Tonstein Band in the lower section of the rib resulted in the increased deformation for the middle and upper rib. Computer modelling was also used to examine the failure mechanisms.

INTRODUCTION

The current risk of rib fatalities in the coal mining industry has prompted further research into understanding the mechanisms of rib failure. Moranbah North Mine (MNM) assisted further research by providing the mine for rib deformation studies as part of the Australian Coal Association Research Program (ACARP) project C25057. This paper provides the results of the rib deformation monitoring project at MNM. MNM is however currently effectively managing rib failure risk through support design and Trigger Actions Response Plans.

An approach of monitoring rib deformation and rib support loads was used to measure and characterise the dynamic rib deformation for mining cycles of development and longwall retreat stress environments. The characterisation of rib deformation, together with rock failure modelling, was used to determine the mechanisms for failure within the rib and to assess the interaction of rib support with the rock failure. The mechanisms of rock failure are considered an important factor in understanding the drivers for the failure. Consideration of the site specific failure mechanisms can provide for a more tailored support design.

MNM is located 16 km north of Moranbah in the Bowen Basin Coalfield in Queensland, Australia (Figure 1). MNM mines the Goonyella Middle Seam (GMS), using retreating longwall extraction methods. The GMS is a thick seam of approximately 5 m to 6 m, leaving approximately 2 m of coal in the roof. The depth of cover for current mining typically ranges from 300 to 350 m.
There is limited literature available on the field measurement and characterisation of the mechanics of rib deformation in Australian coalmines. The majority of public domain data is from industry research and is presented in Fabjanczyk et al. (1992), Gale and Fabjanczyk (1999) and Colwell (2006). A summary of the coalfields, coal seams and mines with this published data is presented in Table 1. One of the key missing seams from this dataset is the GMS. This paper provides a publically available dataset on rib deformation for the GMS in the Bowen Basin.

### Table 1: Summary of Australian coalmines with rib measurement data

<table>
<thead>
<tr>
<th>Basin</th>
<th>Coalfield</th>
<th>Mine</th>
<th>Seam</th>
<th>Data Source</th>
</tr>
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<tbody>
<tr>
<td>Sydney</td>
<td>Southern Coalfield</td>
<td>Westcliff Colliery</td>
<td>Bulli Seam</td>
<td>Fabjanczyk et al, 1992; Colwell, 2006</td>
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<td>Bulli Seam</td>
<td>Fabjanczyk et al, 1992</td>
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<tr>
<td>Western</td>
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<td>Springvale Colliery</td>
<td>Lithgow Seam</td>
<td>Gale and Fabjanczyk, 1999</td>
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<tr>
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<td>Coalfield</td>
<td>West Wallsend Colliery</td>
<td>West Borehole Seam</td>
<td>Colwell, 2006</td>
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<tr>
<td>Bowen</td>
<td>Basin</td>
<td>Wyee State Mine</td>
<td>Fassifern Seam</td>
<td>Fabjanczyk et al, 1992</td>
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<tr>
<td></td>
<td></td>
<td>Oaky North Colliery</td>
<td>German Creek Seam</td>
<td>Colwell, 2006</td>
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<td>Oaky No. 1 Colliery</td>
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<td></td>
<td>Kestrel Colliery</td>
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<td>Colwell, 2006</td>
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<tr>
<td></td>
<td></td>
<td>Crinum Mine</td>
<td>Lilyvale Seam (LV0) (German Creek equiv.)</td>
<td>Gale and Fabjanczyk, 1999; Colwell, 2006</td>
</tr>
</tbody>
</table>

### MECHANICS OF RIB DEFORMATION

In underground coal mining the types of rib failure are typically understood as kinematic failures such as planar, wedge and toppling failures, or stress driven failures such as shear failure and buckling. These failure types can occur in isolation or in combination with each other.

These failure types, however, are often the result of more complex rock failure mechanisms. For example a wedge of rib may not only be driven by discontinuities such as cleat and bedding. The instability of the wedge may be a combination of:

- Discontinuities including cleat and bedding
- Shear stress localised on a non-coal band within the rib acting to fail on the bedding of the non-coal contact and therefore reduce the strength of the bedding
- Vertical stress driven shear fractures creating dilation within the rib to push out the wedge

There are a number of factors that directly influence the rib failure mechanisms and the ultimate rib stability. These key factors include:
• Rib height
• Cleat and joint network
• Mining induced fractures
• Stress (vertical, tributary and abutment loads; 3D stress – $\sigma_1$, $\sigma_2$, $\sigma_3$)
• Coal strength
• Presence of weak contacts (e.g. bedding, claystone bands, mylonite)
• Roof and floor lithology and stiffness
• Reinforcement or skin confinement
• Roof and floor deformation (physically interacting with the rib)

There are also indirect factors that influence rib stability primarily due to their impact on stress distribution and stress driven deformation. Some of these indirect factors include:

• Direction of mining
• Pillar geometry
• Seam depressurisation
• Roof and floor deformation (redistributing stress)

Understanding the drivers of the failure mechanisms is highly beneficial in providing the best rib support design. This may also highlight the current limitations in rib support methodology to proactively support the correct failure mechanism for the ribs.

**MONITORING ARRAY**

A rib and roof monitoring array was implemented to define and measure the progression of deformation in response to the dynamic mining process on both development and longwall retreat. Two monitoring sites were installed. Their locations are presented in Figure 2 and described as follows:

- Development site – MG112 A Heading, 10 m inbye 39 Cut-through, 350 m depth of cover
- Longwall retreat site – mid-pillar MG111 26 Cut-through, 330 m depth of cover

The major horizontal stress direction is approximately north-northeast in the location of LW111 and LW112 creating higher stress concentrations on the cut-throughs. However, the thick seam allows for a coal roof which reduces the stress concentration in the immediate roof.

The instrumentation array in the ribs consisted of sonic extensometers, strain gauged shear strips (measuring vertical shear in the rib) and instrumented bolts. A sonic extensometer was also located in the roof in addition to roof convergence monitoring for the longwall site.

The development instrumentation was installed 3 to 5 m from the face and monitored during roadway advance. The longwall instrumentation array was installed ahead of longwall abutment loading and monitored during the approaching and passing of the longwall. The instrumentation array for the development and longwall sites is presented in Figure 3.
DEVELOPMENT RESULTS

The extensometers showed that minimal deformation was initially observed on development in the first 30 m of continuous miner advance. The measured mid rib depth of softening was 0.8 m to 1.1 m into the pillar side rib only. The lower rib deformation with a depth of softening at 1.7 m into the rib coincided with the location of the Tonstein Band. The pillar side deformation was focused on the mid to lower rib with 15 mm deformation. No deformation was monitored on the lower extensometer of the block side rib, due to damaged extensometer tubing at the location of the Tonstein Band.

The bolt loads, presented in Figure 4, showed high strain zones at the top of the rib, showing the importance of the top rib bolt. The lower rib bolt load and lower rib extensometer showed high strain zones about the Tonstein Band. This suggests that the Tonstein Band, combined with no support at this location is a driver for rib deformation.

During 30 and 200 m of continuous miner advance, the rib continued to deform with greater magnitude of strain in the lower rib. This is observed in the bolt load data in Figure 4, particularly in the lower rib bolts where they show the most significant deformation. In the
case of the lower rib bolt on the block side, these changes were significant enough to destroy the strain gauges on one side of the bolt. The calculation of the bolt loads are an average of strain on opposing sides of the bolt, thus the load and strain results were not able to be calculated in this instance. The shear strip data in Figure 5 also shows a change in the rib behaviour associated with this step change in lower rib deformation. The vertical shear displacement shows the near rib moving in a downwards direction, relative to further into the rib.

An increase in rib deformation beyond the initial dynamic stress during tributary loading, could be due to either a change in stresses or a change in rock properties, typically due to water activity. Given the adjacent longwall was approximately 1.5 km to 2 km from the development site during the monitoring period, stress change is not considered to be related. Therefore the observed movement may indicate clay and water activity occurring on the Tonstein Band to cause deformation well beyond changes in the stress environment.

With the inference that the Tonstein Band has weakened due to a time related change in geotechnical properties, the simultaneous changes observed in the mid-rib are likely to be related to the deformation occurring on the Tonstein Band. It is inferred that the Tonstein Band sheared, in turn reducing the ability for the lower rib to generate confinement, and allowing the mid-rib to deform. Strong roof, floor and bedding contacts in a pillar, or in the rib, increases the pillar's ability to generate confinement and the overall coal strength of the pillar and rib.

Rib spall is often observed about the Tonstein Band at MNM significantly outbye of the initial drivage. The monitoring observations of delayed rib deformation could explain this rib spall experience at MNM.

**DEVELOPMENT INTERPRETATION**

The inherent operational constraints in installing rib support mean that the rib support is typically installed approximately 5 m from the development face. The nature of mining induced fractures about roadways is that they form ahead of and around the corners of the development face. As the miner advances through the fractured ground, some of the deformation in the rib has already occurred before rib support is installed. This may be a reason for the low magnitudes of deformation that were observed in the monitoring data.

Despite this limitation of measuring deformation back from the face, the results showed a key finding in the long term stability of the development roadways at MNM. The step increase in deformation some weeks after the roadway was initially driven, highlights a time component of the rib stability. The focus of deformation about the Tonstein Band indicates that the instability of the Tonstein Band is a driver in the mechanics of rib deformation on development at MNM.

The staged interpretation of the development site deformation is presented in Figure 6 and shows the pillar side rib deformation and bolt loads prior to, and after, the interpreted movement along the Tonstein Band.
Figure 4: Bolt data for the development site
Figure 5: Shear strip data for the development site

Figure 6: Interpretation of development deformation
LONGWALL RETREAT RESULTS

During the retreat of LW111, the ribs and roof in 26 cut-through were monitored for deformation in response to the progressive longwall abutment loading. The monitoring showed substantial rib deformation and minimal roof deformation. The monitoring also showed a stepped progression of deformation whereby the rib deformation suddenly increased in what appears to be the process of shearing of the Tonstein Band. Similar behaviour was observed on the inbye and outbye ribs of the cut-through, however the timing of the deformation stages was delayed on the outbye rib due to the later loading of the outbye pillar. The results for the instrumented bolts, extensometers and shear strip monitoring are presented in Figure 7, Figure 8 and Figure 9 respectively.

The depth of softening on the inbye rib was 1.2 to 1.6 m with a magnitude of displacement of approximately 10 mm until the lower extensometer sheared off at the location of the Tonstein Band. Once the lower extensometer sheared off, the middle and upper extensometers increased in their depth of softening to approximately 3 to 4 m with a corresponding increase in displacement of 45 to 65 mm.

Figure 7: Bolt load data for the longwall site
Figure 8: Extensometer data for the longwall site
A major shear plane was observed in the inbye rib on the instrumented bolts and shear strip at 1.4 m into the rib. This corresponds with the initial depth of softening of the rib, and coincidentally corresponds with the depth of the primary rib support.

The outbye rib showed a depth of softening of 1.5 to 2.6 m with a magnitude of displacement of 20 to 25 mm prior to all the holes shearing off. All extensometers in the outbye rib sheared off between longwall locations of 46 m and 93 m past the instrumented site. The location of the shears increased the depth of softening to 2.1 to 3.4 m.

Both ribs exhibited this step change in deformation. The inbye rib however showed the key information whereby when the lower extensometer sheared off, the increase in deformation and depth of softening was observed in the middle and upper rib. As the outbye rib middle and upper extensometers all sheared at the same time, the magnitude of deformation could not be confirmed in this rib.

**LONGWALL RETREAT INTERPRETATION**

The interpretation of the mechanism of deformation in the rib was inferred from the rib deformation monitoring data. The key element of the shearing of the Tonstein Band, and the step change that it created in the inbye rib deformation, assisted in interpreting the deformation in the outbye rib.

Figure 10 shows the interpretation of the rib deformation due to longwall retreat. The first stage of deformation shows the peak loads, lateral strains and vertical strains with a focus on depth of softening of about 1.5 to 2 m into the rib.
The additional shear stress incurred about the roadway due to the longwall abutment loading causes the Tonstein Band to fail along the bedding plane. Once this plane fails, the lower rib loses its ability to confine the pillar or rib, and the middle and upper rib moves out towards the roadway. This horizontal displacement reduces the vertical stress transfer in the near rib and redistributes it further into the rib. In this case, the stress redistribution created failure of the coal further into the pillar, increasing the depth of softening to 4 m on the inbye rib, and a minimum of 3.4 m in the outbye rib.

The shape of the depth of shearing in the outbye rib shows a greater depth of softening at the Tonstein Band. This supports the inference that the Tonstein Band is a key driver in the mechanics of rib deformation.

**MODELLING ASSESSMENT OF RIB DEFORMATION**

A modelling assessment was conducted to simulate the behaviour of rib deformation to assist in the interpretation of the monitoring results. The models were rock failure models in FLAC 2D using “in house” rock failure routines based on Mohr Coulomb failure criteria. A description of the model process and validation can be found in Gale and Sheppard (2011).

The models were based on site specific geotechnical properties and strata from the closest borehole to the monitoring sites, borehole DDH436. The sonic inferred unconfined compressive strength (UCS) used for model inputs are presented in Figure 11. Models were run for the development and longwall retreat scenarios and roadway geometries. Longwall abutment loads based on empirical data were applied to the longwall retreat model.
Figure 11: Model UCS inputs based on borehole DDH436 sonic inferred UCS

Although there are many model outputs such as rock failure modes, displacements and bolt loads, the incremental shear strain outputs are presented in Figure 12 to show the general distribution of strain within the rib. The location of high shear strain is significant to the location of peak bolts loads, vertical shear and lateral shear observed in the monitoring data.

The development model outputs of incremental shear strain in Figure 12a highlights the high strain zones at the upper corners of the rib and a zone about the Tonstein Band. The majority of rib deformation is observed in the lower rib below the lower rib bolt, where the Tonstein Band is also located. The Tonstein Band strength properties consisted of low shear strength properties of 0.5 MPa cohesion and a friction angle of 10 degrees.

For the longwall retreat model, a sensitivity analysis was also conducted on the geotechnical properties of the Tonstein Band as this is indicated to be a key driver in the total rib deformation. Strong and weak shear strength properties were applied to the Tonstein Band where the weak shear properties appear to match the observed deformation. The strong Tonstein Band properties consisted of bedding cohesion of 2 MPa and friction angle of 20 degrees, while the weak Tonstein Band properties consisted of bedding cohesion of 0.5 MPa and friction angle of 10 degrees.

The two modelled scenarios of strong and weak strength Tonstein Band properties are presented in Figures 12b and 12c, respectively. The shear strain plots highlight the sensitivity of the rib deformation to the geotechnical strength properties of the Tonstein Band. The model with strong shear strength properties in the Tonstein Band shows a general arc shape of rib deformation from the roof to the floor (Figure 12b). With weaker shear strength properties, the Tonstein Band shears along its bedding plane and increases the deformation in the strata above the Tonstein Band (Figure 12c).

It would appear that the deformation of the ribs before the step change is more consistent with the model simulation of the strong Tonstein Band properties where there is minimal shear deformation on the Tonstein Band. This produces a consistent arc shape of depth of softening from the roof to the floor. The observed deformation after the step change creates a shape more closely related to the model where the Tonstein Band has sheared. This produces a depth of softening shape focussed at the Tonstein Band.

The models highlight the mechanisms which create the characteristics of deformation observed at the MNM longwall site. The difference in magnitude between the model and measured deformation could be accounted for with a measured abutment load rather than an empirical abutment load applied in the model.
Figure 12: Rock failure model results showing incremental shear strain to highlight areas of deformation

CONCLUSIONS

Monitoring of rib deformation characteristics throughout the dynamic stress environments of development and longwall retreat provided a significant insight into the mechanics of rib deformation at MNM. The mechanics of rock failure within the rib deformation process highlighted the key driver in the rib deformation to be the shear failure of the Tonstein Band.

The failure of the Tonstein Band in the lower rib then created a step change in rib deformation in the middle and upper rib. This is due to the reduction in lower rib confinement creating a reduction in rib strength.

An understanding of the mechanisms of failure within the rib can be used to investigate various support design strategies, or mining geometries, that may provide for better support of this failure mechanism.

ACKNOWLEDGEMENTS

The author would like to thank ACARP for their monetary funding and the ACARP committee for their project mentoring. The author would like to thank Moranbah North Mine for their monetary and in kind funding for the project, in particular Wesley Noble and Graham Morris for their assistance in managing the operational component of the field work. The author would also like to thank Adrian Rippon from SCT Operations Pty Ltd for his involvement in the installation of instrumentation and data analysis for the rib monitoring program.
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A CONTINUOUS ROOF AND FLOOR MONITORING SYSTEM FOR TAILGATE ROADWAYS

Paul Buddery 1, Claire Morton 2, Duncan Scott 3, Nathan Owen 4

ABSTRACT: Moranbah North Mine has employed the use of Remote Reading Tell Tale Systems (RRTT Systems) since 2012 for the purposes of ensuring accurate, continuous and real time roof movement monitoring for critical infrastructure roadways.

Recently an integrated monitoring system was installed in the tailgate roadway of LW112 to monitor and record the continuous ground movement outbye of the retreating longwall face to better understand both the roof and floor movement. Vertical convergence of the tailgate roadway during longwall retreat is typically managed by installing both active support elements and standing support at Moranbah North Mine. The data has provided significant insights into the magnitude of both roof and floor movement outbye of the longwall which has enhanced the understanding of the required densities of standing support at various locations throughout the tailgate. The system has also demonstrated its potential to be used as a management tool, e.g. during extended face stoppages. Once installed, the monitoring system is entirely automated and the data is automatically collected and transferred to the surface via an optical fibre cable. The system and real-time communication are flexible and can be tailored to meet site specific monitoring needs.

The system includes: an RRTT System, real time convergence monitoring probes and a real time data acquisition, communication and reporting system.

The system enables Moranbah North to be able to measure total vertical roadway convergence and roof displacement continuously without having to access the tailgate at regular intervals. The combined data can be used to investigate: required standing support capacity and density; tailgate roadways ‘zones’ of increased vertical loading associated with intersections; the influence of strata and structural variation; and the optimum support strategy for ensuring roadway stability outbye of the longwall face. The analysis and results produced indicate that standing support densities appear to exceed the required support loads and with continued monitoring providing more data, it may prove possible to optimise support spacings.

INTRODUCTION

The Remote Reading Tell Tales (RRTT) System was initially introduced to Moranbah North Mine following a significant roof failure in the mines entry drift and has since been applied throughout the mine in outbye critical excavations and active production areas. Once installed, the monitoring system is entirely automated and the data is automatically collected and transferred to the surface via an optical fibre cable. Alarms are pre-set to defined triggers and alerts are sent out automatically if triggers are reached. The real time data can be accessed anywhere with available internet connections and log on details.

While historically data collection relied upon personnel conducting physical inspections of conditions or mechanical monitoring devices, the real time communication systems are designed to detect changes and provide notification of abnormal or unexpected roadway behaviour. Confirming alerts in real time can ensure action can be taken immediately and the TARP system can eventually be evolved and redesigned to move from a reactive approach to
a proactive management strategy. The system and real time communication systems are flexible and can be tailored to meet site specific monitoring needs.

Use of the RRTT System in Moranbah North's tailgate roadways has been introduced to provide benefits of reduced personnel exposure to tailgate roadway conditions, to verify support adequacy and provide potential opportunity for optimisation of support densities.

Tailgate roadway stability outbye and alongside the production face is critical to the safe, continued operation of longwall mining. Typically tailgate roadways present limited access to personnel due to respirable dust exposure, elevated gas during cutting and poor roof and rib conditions. The Remote Reading Tell Tales RRTT system are routinely installed in Moranbah North Mine tailgates in order to reduce the exposure time of personnel to carry out district inspections and read mechanical tell tales by providing continuous real time roof movement data which gives an insight into tailgate conditions outbye of the face.

In areas where the tailgate experiences elevated horizontal stress generated during retreat, this has the potential to adversely affect the behaviour of both the roof and floor. While roof movement is captured constantly, to determine the presence and magnitude of floor heave; requires physical observation. Recently an integrated monitoring system was installed in the tailgate roadway of LW112 to monitor and record the continuous ground movement outbye of the retreating longwall face to better understand both the roof and floor movement. With the introduction of real time convergence monitoring, Moranbah North is able to measure total vertical roadway convergence continuously without requiring personnel to access the tailgate at regular intervals.

REAL TIME MONITORING INSTRUMENTATION

The integrated roadway movement monitoring system consists of:

1) 2 - anchor Remote Reading Tell Tales,
2) Real Time Convergence Monitoring Probes (measuring total vertical roadway convergence)
3) Real time data acquisition, communication reporting and recording system

Remote reading Tell Tales (RRTT)

The RRTT system (Figure 1) is an approved Intrinsically Safe (IS) electronic Telltale system which allows real time measurement of roof displacement to a surface computer/anywhere in the world. Each anchor has a coil and ferrite core. Roof displacement causes the coil to move over the ferrite resulting in a change in the inductance which is converted to a displacement. The accuracy is ± 0.1mm. Although 2-anchor RRTTs are currently used, 4 anchor versions are available.

Real Time Convergence Monitoring Probes (RTCMP)

The RTCMP system (Figures 2 and 3) is an approved IS electronic convergence measuring system which, as with the RRTTs, allows real time measurement of vertical roadway convergence to a surface computer/anywhere in the world. They function in the same way as the RRTTs.
Figure 1: RRTT two-anchor schematic

Figure 2: Real time convergence monitoring pogo schematic
COMMUNICATION and INTEGRATION OF THE SYSTEM

The components of the monitoring system are integrated and communicate via the connections shown in Figure 4. The RRTT and RTCMP are installed in the gateroad and connected to an underground data logger which scans connected instruments continuously, recording data at 30 minute intervals and transferring these updates to the mine Ethernet via a Surface to Underground modem.

HISTORY OF MONITORING TAILGATE ROADWAYS AT MORANBAH NORTH MINE

Moranbah North Mine has undertaken several significant monitoring campaigns in the tailgate roadways since 2012, in efforts to better understand the tailgate roof, floor and support behaviour in order to:

- Ensure support adequacy
Consider potential support optimisation strategies
Investigate roadway convergence events

Monitoring campaigns outside standard mechanical telltale and visual inspections were originally started in 2012 in TG109 and involved simple convergence monitoring of Link’n’Lock compression requiring regular physical inspections. Tape measures were secured to installed Link’n’Locks and one end fixed with a record marker nail installed at the other end (Figure 5). As the roadway experiences convergence, the free end of the tape measure would move past the record marker nail and the magnitude of this movement would be recorded by the observer during regular inspections. This movement value recorded in millimetres and then converted to a strain value which, based on laboratory data, could be used to give an estimate of the load on the Link’n’Lock.

![Figure 5: Arrangement for Link’n’Lock strain measurements in TG109](image)

The original spacing of the Link’n’Locks was 3 m centre to centre in headings and 2 m centre to centre across intersections. This was opened up first in headings to 4.5 m centre to centre and subsequently across all intersections. An increase in strain at the face of the order of 0.005 was recorded as a result of increasing the spacing, i.e. headings from about 0.014 to 0.019 and intersections from about 0.019 to 0.024. The maximum recorded strain value (0.024) suggesting a total load at the face of no more than about 150 tonne and the difference of 0.005 represented an increase in load of no more than 30 tonne. The study provided sufficient information to carry out a section of retreat successfully, without the use of routine standing support installed. On the basis of this initial monitoring trial, further work was done on alternative secondary support designs LW601 and LW204 without the routine use of standing support, but with increased tendon support. As part of the ongoing monitoring strategy, simple convergence pogos were used to measure roadway convergence where standing support was not installed and therefore unable to be measured. Figure 6 shows strain data from the Link’n’Locks in LW109 and convergence pogos in LW601 and LW204. It can be seen that there was significantly less convergence in the latter two LWs although it should be noted that they were shallower than LW109, about 60 m in the case of LW601 relative to LW109. This strategy was employed for mining of the subsequent longwalls 110, 602, 111 and the current longwall block, 112 (Figure 7).
THE USE OF REAL TIME MONITORING FOR TAILGATES AT MORANBAH NORTH MINE

Initially, the geotechnical information collected for the tailgate standings support study came from mechanical Tell Tales and roadway inspections and observation. However, the significant investment of personnel resources and the requirement for regular access to the tailgate to gather data such as this prompted the extensive use of a continuous and remotely accessible monitoring system. The introduction of the RRTT System in tailgate roadways commenced as a restricted trial in the tailgate roadway of LW601, which was mined directly following LW109. The trial had several objectives including: proving the viability of the RRTT system; reducing the requirement for personnel to access the tailgate roadway, thus reducing the amount of exposure to adverse environmental conditions; facilitating continued production due to reduced downtime to allow tailgate access for personnel and providing the real time
data to help to make decisions on production activities that could potentially impact tailgate serviceability, e.g. the viability of planned stoppages.

LW601 proved to be a successful trial of the viability of the RRTT with the expected benefits listed above being realised. The monitoring information can be observed at the MG Drive by longwall personnel as well as on the surface in real time, providing for transparency for decision making.

The installation of RRTT has become an integral part of tailgate monitoring at Moranbah North Mine. An example of the data which is gathered and made available is presented in Figure 8.

**Figure 8: Comparison of Telltale movement for intersections of TG112**

**OPERATIONAL USE OF MONITORING RESULTS**

The details below provide an example of how the real time data can be used to assist operational decisions:

- 13th of October, a 2 anchor RRTT at 22ct (Figures 9 and 10) alerted of significant roof movement in the intersection. Triggering a rate of movement Trigger Action Response Plan (TARP) on the 8 m roof anchor.

- From the continuous monitoring at Moranbah North mine it was possible to track the rate of movement and complete a comparison against recent movement in inbye cut throughs. Movement rates indicated anomalous and early roof movement in relation to the longwall face position.

- The movement curve was similar only to increased roadway spans such as 4-way intersections, e.g. 25ct monitoring data which recorded early and higher rates of movement due to being a 4-way intersection.

- This early indication which was triggered at only 2.4 mm of movement, gave an opportunity for remote monitoring of the movement rates in real time so that decisions could be made over access to the tailgate by personnel and continuing operations to mitigate the impact of increased rates of roof movement.

- Rates of movement resulted in a goaf overrun for 9 m which was able to be predicted prior to occurring and the impact mitigated.
INTEGRATED MONITORING DATA

During retreat of the later 100 series panels, significant levels of floor heave accompanied by loss of rib-side stability have been experienced, on some occasions resulting in roof fall. As a result, RTCMP were installed concurrent with RRTT in order to determine the magnitude of floor heave resulting in roadway instability. Data from RTCMP installed at 17.5ct and 17.25ct (Figure 11) correlate to earlier convergence data that indicates that the highest levels of roadway convergence occur approaching intersections where higher stress loading is present. Floor movement is occurring further outbye of the face than roof movement alone when comparing RRTT and RTCMP data.

FUTURE PLANS FOR CONTINUED MONITORING

While in its infancy, the integrated monitoring system engaged in TG112 has provided beneficial, useable and relevant data in a short period of time. Due to the nature of the
system – delivering data in real time and continuously, it has also offered the opportunity to observe the absolute magnitudes of roadway convergence and provide ongoing movement feedback so that time critical decisions can be made around personnel access to the tailgate, planning of maintenance or retreat strategies. The system has the potential to increase understanding of the critical geological and geotechnical conditions that impact on tailgate roadway serviceability and provide alerts in real time for proactive strata control management. Some critical parameters that are yet to be quantified to continue to improve control of tailgate roadways to ensure continued serviceability and minimal production impact are:

- Understanding the magnitude of floor heave occurring due to the presence of weak floor strata associated with lower seams coming in closer proximity to the Goonyella Middle Seam.
Further understanding of the high strain zones identified above the working seam or near to the face. Information such as this could assist in better developing hazard zoning for the purposes of support design and refining scheduling for longwall retreat.

- Early and rapid warning of deteriorating conditions – particularly around tailgate intersections or when an excessive roof span develops between the tailgate chain pillar rib and the last longwall powered support.

SUMMARY and CONCLUSIONS

The evolution of roadway monitoring at Moranbah North Mine has provided many interesting results. The development of reliable real time monitoring systems has successfully monitored roadway stability and provides valuable data for ongoing support design while keeping people out of harm’s way.

ACKNOWLEDGMENTS

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DENDROBIUM MINE LONGWALL PRE-DRIVEN RECOVERY ROADS INCLUDING DEVELOPING CUTTABLE GROUT PILLARS

Matthew Johnson¹, Rob Thomas², Verne Mutton³ and Michael Egan⁴

ABSTRACT: Longwall face recovery is a complex geotechnical challenge with high potential for losses due to the occurrence of ground control problems. In recent times, the use of pre-driven recovery roads has improved the overall efficiency and safety of moving longwall equipment from panel to panel.

In 2015 Dendrobium Mine and Golder Associates developed a grout pillar design that could be constructed within the pre-driven recovery road prior to the holing and recovery of the longwall equipment. The challenging ground conditions often experienced in longwall pre-driven recovery roads provided the main impetus for the decision to construct large yieldable grout pillars in a roadway that will be subjected to a cantilevering roof and a yielding fender on holing. Due to the resulting increase in support density, recovery road conditions have improved as shown by roadway monitoring data. Longwall operators have considerably less exposure to potential hazards during bolting operations with reduced bolt up time resulting in increased longwall productivity.

All elements of support including 47 grout pillars had to be safe to cut and remove via the mine conveyor system. Due to lack of surface access, all materials to construct the grout pillars were transported up to 14 km from the mine portals. Grout pillar formwork design initially incorporating fabric bladders and steel formwork has since been replaced with a lattice of fibreglass (FG) dowels and hessian coated FG mesh, sprayed with a Gypsum based plaster. Air-driven placer pumps were replaced by electric placers with greater reliability and grout quality control procedures ensured that the target pillar strength was reached before holing of the longwall.

Pre-driven recovery road support design incorporating cuttable grout pillars has been combined with the development of a grout delivery system and containing formwork to deliver key productivity and safety improvements during longwall recovery.

INTRODUCTION

Dendrobium Mine extracts 300 m wide longwall panels (partial mine layout shown in Figure 1) from the lower 3.7-4.1 m of the Wongawilli Seam. In the current area of operation (Area 3B), the Wongawilli Seam is approximately 9-10 m thick. Working in the lower section of the seam leaves approximately 5 m of weak coal and shale material in the immediate roof. The weak roof and thick seam longwall environment make the longwall face prone to cavity development, particularly during periods of slow retreat where convergence promotes the formation of fracturing ahead of the face. Cavities commonly reach the top of the seam and have the potential to cause significant operational delays. Cavities have proven to be the most difficult to manage from the point the recovery mesh has been pinned to the final longwall recovery.

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Longwalls 6-10 at Dendrobium Mine were recovered using traditional bolt-up methods. Longwalls 11 and 12 have been recovered using a Pre-Driven Recovery Road (PDRR). With traditional bolt-up, a recovery mesh sheet was pinned to the roof approximately 15 m from the...
final face position with subsequent pinning towards the recovery road with nine rows of 170 roof bolts (one per support) installed using rapid face bolters (see Figure 3). Total support installed on this critical path consisted of approximately 1500 x 2.4 m roof bolts, 170 x 8 m cable bolts and 500 x 1.8 m rib bolts. This support method with the bolted recovery mesh proved difficult and time consuming to install in cavity roof conditions. Recovery mesh was commonly damaged in the process and a high degree of operator awareness was required to install the supports safely. Large cavities carried into the final recovery position often resulted in the difficult recovery of longwall supports and the requirement for additional ground consolidation and cavity fill.

Figure 3: Typical Dendrobium bolt up arrangement (Longwalls 6-10)

There was a significant increase in the time taken from mesh on to chain break from Longwall 6 recovery onwards. The increased difficulty of bolting up the longwall face coincided with the move to Area 3 and associated increases in depth of cover and longwall abutment loading. Mesh installation to chain break averaged 9.5 days for Longwalls 1-5 and 28.4 days for Longwalls 6-10. Longwalls 11 and 12 recovered using a PDRR have averaged 4.8 days. (See Table 1 and Figure 4)

Table 1: Summary table for longwall face recovery

<table>
<thead>
<tr>
<th>Longwall Panels</th>
<th>Depth of cover at recovery point (m)</th>
<th>Average time taken from mesh on to chain break (days)</th>
<th>Panel Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-5</td>
<td>160-260</td>
<td>9.5</td>
<td>240</td>
</tr>
<tr>
<td>6-10</td>
<td>270-370</td>
<td>28.4</td>
<td>240, 300 m from LW8 onwards</td>
</tr>
<tr>
<td>11-12</td>
<td>360-390</td>
<td>4.8</td>
<td>300</td>
</tr>
</tbody>
</table>
Following the poor recovery performance of Longwalls 6-10 the decision was made to pursue alternative options and Dendrobium Mine engaged Golder Associates and Minova to develop solutions. Several concepts were entertained including; a full face pre-driven recovery roadway (PDRR), partial pre-driven recovery roads and access roadways to allow for pre-consolidation of high cavity risk areas of the face (mid-face). Once it was demonstrated that the full face pre-driven recovery for Longwall 11 was technically feasible and operationally achievable in the scheduled time available, it was chosen as the course of action. The partial PDRR method was discounted due to concerns that slow retreat through zones prone to cavities in areas of the face with no PDRR would lead to deterioration of the fender between the face and the partial PDRR. Access galleries and pre-consolidation were discounted due to the high cost and uncertain outcome.

SUPPORT DESIGN

It is generally agreed that some form of standing support should be installed in roadways that will for various reasons, be holed through with a longwall. Considering the magnitude of the associated abutment loading in conjunction with the need to ensure that the material used is both strong and cuttable, some form of cementitious support is often used.

By far the most common type of standing support used in pre-driven roadways is some form of fibre-crete block; although a number of operations have used pumpable grout filled cribs, cement blocks or backfill; where in the case of the latter, the full cross-section of the roadway is filled with either a cement-flyash mix or cellular concrete. There are however a number of significant deficiencies associated with these cementitious supports; namely the strength and yielding ability of pumpable cribs, the need to ensure that fibre-crete or cement based supports are softened with timber such that they are able to yield in a controlled manner, the slenderness of the supports such that they are able to control any out-of-plane loading that may result on holing, and the cost and downstream impact on the conveying equipment associated with large volumes of backfill material.

An assessment has addressed the design and use of grout filled pillars, which not only offer a high capacity and squat form of standing support, but also do not necessitate the use of timber and the large volume of material and the associated infrastructure typically associated with backfill. Similarly, the placement of the grout pillar against the inbye rib, confines and in doing so maximises the strength of the fender on holing, and the strength and yielding ability of the grout pillar can be further increased through the use of mesh and/or fibre-glass bolts (Figures 5 and 6).
The study indicated that grout pillars offer several of the benefits typically associated with backfill, in particular the use of a high capacity, squat and cuttable form of support that had the added benefit of confining the inbye fender on holing. Critically however, the pillars do not necessitate the large volumes of material associated with backfill, and the large number of supports and timber typically associated with the more traditional forms of cementitious standing support.

In order to maximise stability of the fender, using PDRR and standing support elements it was decided that the PDRR would be driven at the smallest dimensions practical with Dendrobium’s current JOY 12CM30’s miner bolters. As a result the PDRR was driven 5.2 m wide and 2.8-3 m high.

![Figure 5: Longwall 12 PDRR grout pillar, original style grout pillar. Constructed with rubber bladder and removable steel formwork](image1)

![Figure 6: Longwall 13 PDRR new style grout pillar, cuttable form](image2)

The geotechnical environment and required grout pillar and bolted support densities were divided into two distinct zones; protected ends located at either end of the roadway and an intervening mid-face area.

Longwall support convergence and leg pressures were reviewed to determine the extent of the protected end areas. Based on shield loading data it was demonstrated that less vertical abutment load could be expected within 30-50 m of each gate road. The protected ends were conservatively assessed to be 30 m from the block side rib in both the tailgate and maingate of Longwall 11 PDRR.

In regard to the specifics of the installed standing support, it is of note that the pillars were filled with grout that was required to attain a minimum strength of 7 MPa on holing. Furthermore, in LW 11’s PDRR a minimum standing support density of 3 MPa was installed in the mid-face area and a minimum standing support density of 2.2 MPa in the protected ends; this decision being based primarily on the considered high likelihood that the longwall could (as per the neighbouring longwalls) experience some level of periodic weighting on holing. As a point of reference in this regard, it is also of note that recent experience in longwall pre-driven recovery roads suggests that unless there is some relevant precedent that indicates otherwise, the mid-face area in the pre-driven recovery road should be supported with a standing support density of at least 1.5 MPa and the protected ends, at least 0.8 MPa.

Following the successful precedent of LW 11’s holing, and accepting that the maingate end of LW 12’s PDRR was driven through a significant thrust fault, it was subsequently decided to maintain the 3 MPa density of standing support in the fault affected protected end and mid-face area, and reduce the density of standing support in the remaining mid-face area to between 2.2 and 2.6 MPa and the un-faulted protected end to 2 MPa.
Other points of note with regard to the installed support include (i) the decision to reinforce the grout pillars with 1.8 m long fibre-glass bolts and Tensar Mesh, (ii) the installation of fully grouted Megadowels to help reinforce the fender and the outbye rib on holing, (iii) the use of 8m long Sumo cables in the roof to help maintain some form of beam action in the roof on holing and (iv) a combination of low angled spiles and twin-stand cables over the fender to help both reinforce the roof on the face on holing and secure the goaf edge during the recovery of the shields (for details, see Table 2 and Figures 7 and 8). Figure 7 shows a plan view of the layout of the 47 Longwall 11 PDRR grout pillars constructed against the longwall fender.

### Table 2: Support density installed in LW 11’s PDRR

<table>
<thead>
<tr>
<th></th>
<th>Roof support</th>
<th>Fender rib support</th>
<th>Fender roof spiles</th>
<th>Outbye Rib Support</th>
<th>Standing Support</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Protected Ends</strong></td>
<td>2 x 60 t 8 m cables/m, 8 x 2.4 m roof bolts/m</td>
<td>2 x 4 m 40 t cuttable dowels/m with cuttable mesh</td>
<td>1 x 6 m twin strand and 1 x 6 m 28 mm coupled thread bar per 1.75 m</td>
<td>3 x 4 m 40 t cuttable dowels/m with steel mesh</td>
<td>1 x 3.7 m x 4 m 7 MPa grout pillar at 8 m centres</td>
</tr>
<tr>
<td><strong>Mid-face</strong></td>
<td>2.5 x 60 t 8 m cables/m, 8 x 2.4 m roof bolts/m</td>
<td>3 x 5 m 40 t cuttable dowels/m with cuttable mesh</td>
<td>1 x 6 m twin strand and 1 x 6 m 28 mm coupled thread bar per 1.75 m</td>
<td>3 x 4 m 40 t cuttable dowels/m with steel mesh</td>
<td>1 x 3.7 m x 4 m 7 MPa grout pillar at 6 m centres</td>
</tr>
</tbody>
</table>

**Figure 7: Longwall 11 PDRR grout pillar layout**
DEVELOPMENT OF A CUTTABLE GROUT PILLAR FOR RECOVERY ROAD SUPPORT

As with all recovery road support elements within the longwall cutting horizon, the grout pillars used for supporting the recovery road during holing were required to be safely cut by the shearer to be loaded onto the conveyor system. Due to surface access restraints resulting from mining beneath the water catchment it was necessary to transport all grout and formwork materials from the surface portal, approximately 14 km to the recovery road sites for construction. This posed logistical and storage challenges underground.

It must be noted that there has to be sufficient mine development float in order that a recovery road can be supported with grout pillars. The time from grout manufacture and delivery to grout filling of pillar formwork, along with waiting for sufficient strength development of the last completed pillar, can take up to 6 months. Minova were asked to provide a suitable grout and pump delivery system that would reliably mix and pump grout infill for the 45 pillar supports. This system has been developed from experience gained in pillar support of the initial recovery road in Longwall 11. It also resulted in the trial and development of a new pillar formwork method and design and manufacture of electric placer pumps. A list of design guidelines critical to the success of the project was provided by the mine as follows:

1. To be able to pump one pillar (footprint 4 m length parallel to the fender, 3.7 m depth and roadway height) of approximately 50 m$^3$ per 24 hour period;
2. The pillars had to be pumped from a distance of up to 350 m;
3. Delivery of the grout from the factory on specific schedule;
4. The initial pillar formwork consisted of heavy steel framework and fabric bladders;
5. The grout uniaxial compressive strength (UCS) at 28 days was to be in the range 7-10 MPa;
6. The grout pillars could withstand the load from vertical abutments right up to the last shear;
7. The grout was to be transported in bulk bags and the bag height was to be lowered in order that bags could be placed over the pumps using an overhead monorail.

Development and improvement of the grout delivery method and containment for pillar construction has occurred over three recovery roads from Longwalls 11 to 13. The genesis of improvements to pillar construction since initial discussions started on the recovery road pillar project in September 2015 will be included within the following topics.

High yield grouts

FB200 high yield grout was chosen for the pillars as it is dimensionally stable and able to be 1) self-supporting due to its rapid strength gain and 2) accept strata convergence without...
rapid loss of strength. The mine had previous experience with using FB200 for roadway fall consolidation.

FB200 is able to penetrate into the smallest voids due to a low viscosity ≈100cP before gelation. This grout has been used for consolidating backfill to create artificial pillars for sill pillar recovery, building bulkheads including explosion rated ventilation seals and consolidation of fallen roadways via surface borehole delivery to enable re-mining and roadway recovery.

Ettringite (CaO₂Al₂O₃·SO₃·32H₂O) is formed as part of the chemical process (Mutton, et al., 2010) with Calcium Alumino-Sulphate (CSA) cements in the hydration and formation of FB200. This provides a high yield fast setting grout with a low viscosity, able to be placed at a water/powder ratio of between 1:1 and 4:1. The dimensionally stable grout is able to accept high strains.

Development of a grout sampling method

In order to achieve the required 28 day strength, FB200 was pumped at a water to powder (W:P) ratio of 2:1. Longwall 11 recovery road pillars were initially pumped with pneumatic placers. The air supply pressure is affected by in-bye operation of equipment such as air-track bolting rigs, so that it is necessary to regularly check and adjust water flow to the placer. For this reason a method was developed to check and maintain the desired W:P ratio ensuring a grout strength requirement of 7-10 MPa at 28 days. Two 100 mm polystyrene cube mould samples were taken for each pillar for quality assurance laboratory testing. Average grout UCS at 28 days was 8.38 MPa.

Close control of grout density to a value of 1.26 was achieved by weighing 100 mm polystyrene cube samples with a 5 gram accurate 0-2 kg mechanical bench scale (Cube weight is 30 grams). Water flow to the pump was adjusted if the density fell outside the specified range. The staged procedure for FB200 density check has the following actions.

Normal operating density range of 1.26 – 1.30.

If sample weight [kg] lies between the limits >1.24, <1.26 and > 1.30, <1.34 then recalibrate the pump and prepare a new sample for weighing.

STOP PUMP. Check the operation of the pump and clean down. REPORT the low density result to the Mine Management. Recalibrate the pump and prepare sample for weighing.

Development of an electric placer pump

Because of concern over the available air pressure 14 km inbye the surface compressors, a 150 nominal bore ring main was constructed to reduce pressure losses for Longwall 11 recovery road pillar construction. However, pump rates varied considerably with air pressure ranging from 310 – 448 kPa (45-65 psi) within the duration of a shift and on this basis, a decision was made to develop electric placer pumps for LW12 pillars. Three pneumatic placer pumps were converted to 1000 volt electrical operation. Extensive redesign resulted in the placers electrical supply compliant with AS/NZS 4871 Electrical equipment for mines and quarries General requirements and AS/NZS 2290 Electrical equipment for coal mines-Introduction, inspection and maintenance, suitable for use in the non-hazardous zones of an underground coal mine.

One major improvement was to develop a two-speed drive train with a lower speed powder feed to be used when manually handling 20 kg grout bags and a high speed for bulk delivery of up to 3 tonnes per hour of dry powder. The Original Equipment Manufacturer (OEM) provided a safety dossier that also prescribes specific maintenance requirements. It was then possible to obtain a sign-off for electrical and mechanical components by suitably qualified Queensland Registered Professional Engineers (QRPE). Figure 9 shows a redesigned placer pump with larger fork-tyne pockets, Rudd lifting lugs, protected electrical boxes (high and low
voltage) and rerouted air and water reticulation. Pump trials with the client were conducted to demonstrate placer operation including pumping distance and throughput.

![Figure 9: 1000 volt electric placer pump side view](image)

**Figure 9: 1000 volt electric placer pump side view**

**Bulk handling and storage of grout**

Longwall 13 required the transport and storage of 650 pallets of FB200 and 250 pallets of Sprayplast™ underground before pumping proceeded. Due to the available pit top footprint, there was limited covered surface storage. Bulk bags were transported underground on their pallets to keep the grout out of the mud. With one dedicated loader and operator per shift, it was possible to transport 30 x 1.2 tonne bulk bags in a 24 hour period using a double-axle trailer with a five bulk bag capacity.

![Figure 10: Double bag QDS carrier](image)

**Figure 10: Double bag QDS carrier**

During the first recovery road pillar construction, approximately 5% of product was being wasted during transport because of bags being ripped or being speared with fork-tynes. Improvement opportunities were identified with the packaging arrangements and how the loader would pick up the bags. A different style of bulk bag was sourced which had internal baffles and a lower profile. These helped the bulk bag sit squarely on the pallet providing a lot higher resistance to toppling during transport. The second improvement to packaging came after the introduction of the double bag handler, a load haul dump attachment allowing transport of two bulk bags on the front of the loader with the bags being supported by suspension straps. This attachment removed the issue of tynes spearing bulk bags and also provided additional support to prevent bulk bags toppling off the pallet during transport.

Two additional straps that were inserted through each pallet also secured each bulk bag to maintain stability. When lifted with the double bag handler, the pallets remained attached to the base of the bulk bags. As such no water or mud could build up on the bottom of the bulk bag once placed in-line with the monorail. With these handling improvements, wastage during transport greatly reduced to ~1%. Figure 10 shows a Quick Detach System (QDS) double bag carrier which was used underground.
Glass Reinforced Plastic (GRP) cuttable formwork method

Longwall 11 recovery road pillar formwork consisted of heavy steel framing and fabric bladders which provided operational difficulties. For longwall 12 a decision was made to trial two GRP dowel lattice and Sprayplast™ pillars located at No 1 position on the tailgate four-way intersection and in the middle of the recovery road at pillar location 22. The tailgate pillar was extended to reinforce the corner of the longwall block. These pillars had GRP threaded bar in an interlocking lattice design supporting inside sheets of 4 mm GRP Powermesh™ covered in hessian cloth.

Once vertical dowels are installed between the roof and floor, three evenly spaced horizontal layers of connected dowels are wrapped around the outside and tied to the vertical dowels. In addition, the formwork is reinforced internally in both horizontal directions each with nine 22 mm FRG dowels and 200 mm diameter retaining plates. Fender rib dowels are resin grouted to anchor the lattice formwork. Each pillar was sprayed with Sprayplast™ Gypsum based plaster using the dry shotcrete application method. 5 to 6 pallets (1.12 tonnes each) were required for each support pillar with all edges in contact with the fender sprayed out 500 mm to lengthen the potential leakage path for the grout.

Location 22 pillar was 4.2 metres in height and was pumped successfully in one pass, whereas previous pillars required 3 separate lifts. Figure 11 shows the Tailgate trial pillar after being sprayed with Gypsum based plaster.

The trial pillars stayed intact right up to the last cut by the shearer as shown in the photograph of Figure 12. Due to the success of the trial pillars, a decision was made to construct GRP and Sprayplast™ formwork for all Longwall 13 PDRR grout pillar supports.

The GRP and Sprayplast™ formwork has the following characteristics and advantages.

i. Each pillar’s formwork can be constructed in 12 hours;
ii. No formwork strip time is required;
iii. Each pillar can be pumped to the roof in one pass;
iv. All pillar formwork can be constructed in one campaign before grout filling;
v. Total weight of each pillar GRP formwork is ~ 150 kg. There is a considerable reduction in manual handling of formwork and also Gypsum based plaster because it is supplied in bulk bags;
vi. Formwork is able to conform to roof cavities, broken ribs and be extended to reinforce pillars;
vii. Cost effective, rapidly installed pillars to reduce construction time;
viii. Grout will penetrate and reinforce the rib fender coal;
ix. The horizontal GRP dowels and plates reduce the bending moments on the vertical dowels from pressure of the pumped grout. The GRP dowels anchored with polyester resins into the fender coal provide stability for the whole pillar;
x. The grout will yield when subject to roof to floor convergence from abutment loads, retaining most of its strength before being cut through by the longwall shearer.
Pumping of a high yield grout

The placer pumps are capable of pumping FB200 grout at a water: powder ratio of 2:1 for 450 metres horizontally. The placer Monopump maximum delivery pressure is 12 bar (1200 kPa) requiring 16 bar (1600 kPa) working pressure 40 mm Polyline to be used for grout delivery. The pneumatically driven monorail system used for suspending and positioning the bulk bags above the placers requires a minimum working height of 4.5 m. As the bulk bag empties of powder it relaxes and lengthens. Excavation of the floor is often required to gain sufficient clearance for the suspended bag over the placer dry powder hopper.

EXPERIENCE HOLING THE RECOVERY ROADS

Longwall 11 and 12 PDRR have been successfully holed and support of Longwall 13 PDRR is nearing completion at the time of writing:

- The roadways were monitored with the following instrumentation;
- Roof to floor convergence pogo stick (see Figure 13).
- Rib to rib convergence stations
- Hydraulic borehole stress cells installed in the grout pillars
- Roof extensometers
- Daily geotechnical inspections were conducted over the last pillar of longwall retreat with the frequency increased to each shift within 50 m of holing the roadway (see Figure 14).

![Figure 13: Longwall 11 PDRR monitoring station – 40 m to hole](image1)

![Figure 14: Longwall 11 PDRR grout pillar with plastic sheeting removed – 40 m to hole](image2)

Very little deformation was visually observed until approximately 25-30 m from holing. At this point roof to floor convergence began to accelerate as abutment load concentrated on the fender. Deformation in the PDRR was primarily seen as floor heave and loading of the fender ribs and outbye ribs.

Sharp increases in roof to floor convergence rates were measured with 10m and 16m wide fenders in Longwall 11 and Longwall 12 PDRR respectively (see Figure 15). At this point the grout pillar supports were working to confine the fender and restrict floor heave. Bulge of the fender rib began exerting the out of plane loading that would traditionally compromise slender standing support elements. Floor heave primarily occurred in the unsupported span of floor between the pillars and between the grout pillars and the outbye rib. As convergence continued the grout pillars started to yield as shown in Figure 16, with failures up to 0.5-1 m deep into the 3.7 m x 4 m pillars observed. Confinement of the grout pillars was improved on Longwall 12 with the addition of a more substantial cuttable mesh wrapping of the pillars as shown in Figure 16.
The PDRR was developed at 3 m high from the 2nd Machine Band down to a coal floor (see Figure 2). This resulted in approximately 0.8-1 m of coal being left on the floor in the roadway (see Figure 2). On the longwall face side the roof horizon and extraction height was lowered to the 1st Machine band at a height of 3.7 m. This allowed the shearer to cut in under the steel roof support installed in the PDRR.

As the longwall holed the roadways the 0.8-1 m coal bench and the grout pillars were cut out, resulting in a final roadway height for recovery of approximately 4 m. Face conditions on the longwall from the mesh pinning row onwards were typically good. Particularly from the point that the countersunk spiles and twin strands were present in the immediate roof horizon. Face stability over the last 5 m of retreat was aided by a high density of 5 m cuttable dowels installed into the fender. Confined by the grout pillars the fender stood until holing where it was cut away to reveal the PDRR. Figures 17 and 18 show holing Longwall 11 PDRR.

Mining through the cuttable support elements in the grout pillars and fender was a successful process that required manning of key coal clearance points to avoid grout pillar cuttable formwork and plastic mesh from blocking chutes and transfers (see Figures 17 to 18). Downstream, the cuttable support elements impacted the coal washery and resulted in additional maintenance works being conducted.

The lack of a horizon marker for the lead shearer drum resulted in some challenges with horizon control that will be managed with improved use of automation from LW13 PDRR onwards. Figure 19 shows the condition of one of the grout pillars in Longwall 12 PDRR.
CONCLUSIONS

The use of pre-driven recovery roads, incorporating large cuttable grout pillars on Longwall 11 and 12, successfully reduced the duration of bolt up by an average of 23 days compared to Longwall 6-10. Longwalls 6-10 were recovered using traditional bolt-up techniques in a comparable geotechnical environment. The continued refinement of the support design, grout pillar construction and pumping technology have made supporting these roadways a repeatable and efficient process that the mine is likely to pursue for the foreseeable future. Longwall 13 PDRR support is nearing completion, Longwall 14 PDRR has been developed awaiting support and Longwall 15 PDRR is currently being developed.

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A REVIEW OF IN SITU STRESS MEASUREMENT TECHNIQUES

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ABSTRACT: Changes in stress orientations and magnitudes can have a significant adverse impact on mining conditions such as increasing the risk of violent failures. Knowledge of these changes can indicate the high-risk zones within a mine site, which will enable mine operators to implement appropriate controls. At deep underground excavations, there are some difficulties in collecting reliable data at reasonable costs and majority of methods provide point measurement per test only. Thus, the utilisation of borehole techniques has received more attentions. In this paper, traditional stress measurement techniques are reviewed, including their pros and cons. Under specific geological conditions, some methods have significant advantages over others. Following the illustration of benefits and shortcomings of these techniques, the development potential of an in situ stress measurement technique using borehole breakout is briefly addressed in conjunction with the future research plan.

INTRODUCTION

In situ stress magnitudes are always of great interest of mining and geotechnical engineering as they are essential to underground operations. With knowledge of the stress field, engineers can identify high risk zones and implement appropriate controls methods to prevent catastrophic failures. A series of in situ stress measurement techniques have been developed to interpret stress magnitudes in different geological conditions at a given point. However, a handful of point measurements sometimes might not be representative for the whole operation as stress field can vary significantly with various tectonic settings and overburden pressure at different depths. At deep underground operation, majority of techniques suffered from collecting reliable data at reasonable costs.

To effectively monitor ground conditions and make wise engineering decisions, it is crucial to obtain a clear understanding of applicability and limitations of each measurement technique. In this paper, various in-situ stress measurement techniques are reviewed with advantages and disadvantages, particularly emphasise on hydraulic fracturing, overcoring and borehole breakout due to their prevalence. Given the rapid advancement of borehole imaging technology, using borehole breakout to estimate stress magnitudes has received much more attention (Gaines et al., 2012). In general, breakout happens at different depths of a borehole and data can be collected with one measurement. This would considerably save time and cost compare with other techniques. It is also clear that breakout dimensions are stress dependent (Zoback et al., 1985; Barton et al., 1988) and some methods have been proposed to constrain stress magnitudes (Zoback et al., 2003; Chang et al., 2010). Therefore, borehole breakout has a great potential to be developed as a primitive stress estimation technique.

In addition, this paper provides a brief discussion about the future research plan of authors in borehole breakout area. Experimental studies have been undertaken; it is expected to be combined with field and numerical data for further analysis.

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HYDRAULIC METHODS

Hydraulic fracturing is one of the commonly used methods for stress measurement, especially in petroleum engineering. Currently, there are two major categories, which included conventional hydraulic fracturing and Hydraulic Tests on Pre-existing Fractures (HTPF).

Conventional hydraulic fracturing

Conventional hydraulic fracturing utilises the hydraulic pressure created by fluid injection to form tensile fractures around the borehole to estimate the in situ stress. It was initially used for the reservoir productivity stimulation and was applied to stress measurement in early 1960s.

A sealed section of the borehole is isolated by a straddle packer. Fluid is then slowly injected into this interval to apply pressure on borehole sidewall. When the pressure induced tangential tensile stress surpasses the tensile strength of the surrounding rock, two fractures occur in opposite directions and penetrate along the plane perpendicular to the minimum principal stress direction, which has the least resistance. This pressure is recorded as breakdown pressure, $P_b$. After two fractures are initiated, there is a pressure required to hold the fractures open and allow fluid flow into them. As postulated by Hubbert and Willis (1957), if the borehole is drilled vertically, the vertical principal stress is the maximum principal stress; then this pressure is equal to the minimum horizontal principal stress magnitude, which can be measured by turning off the injection system, namely, Instantaneous Shut In Pressure (ISIP).

\[
S_h = S_{ISIP}
\]  

(1)

where $S_h$ = minimum horizontal principal stress and $S_{ISIP}$ = instantaneous shut in pressure. Thereby, based on the Kirsch solution (Kirsch, 1898), the maximum horizontal principal stress, $S_H$, can be obtained from Equation 2.

\[
S_H = 3S_h + T - P_0
\]  

(2)

in which $T$ = tensile strength of rock and $P_0$ = pore pressure. The illustration of hydraulic fracturing is displayed in Figure 1. To conduct the stress measurement where tensile strength data is unavailable, Bredehoeft, et al. (1976) modified the conventional equation:

\[
S_H = 3S_h - P_r
\]  

(3)

where $P_r$ = re-open pressure. This is measured by turning on the pumping system cyclically, which enables the fluid to re-open fractures multiple times at the borehole wall. However, due to some uncertainties, such as plastic behaviours of the surrounding rock (Rutqvist, et al., 2000) and remaining apertures at the start of each cycle (Cornet, 1993), the measurement of re-opening pressure may not be accurate. For this reason, the application of this modified equation is limited.
Conventional hydraulic fracturing provides a simple way to measure stress magnitudes, advance knowledge of rock properties is not essential, such as Young’s modulus and Poisson’s ratio. This method also offers a reliable and direct measurement of minimum horizontal principal stress at an accuracy of ± 5% (Ljunggren, et al., 2003). Another advantage of hydraulic fracturing is its utilisation at deep location. Hung et al. (2009) successfully conducted this measurement below 1km depth. With conjunction of ancillary equipment, including impression packer, compass and borehole scanning technique, it is also possible to obtain the approximate stress orientations. This is because the direction of fractures’ propagation is perpendicular to the minimum principal stress direction.

One inevitable shortcoming of this technique is the accuracy on maximum horizontal principal stress calculation. The variation of estimation can be over ± 20% (Ljunggren, et al., 2003). When there are pre-existing weaknesses around the testing section, the injected fluid would re-open and penetrate through pre-existing fractures instead of the plane parallel to the minimum principal stress direction since the resistance is much lower along pre-existing fractures. The disturbance of pre-existing weaknesses would lead to unreliable estimation of the horizontal stress field. Hence, this technique is not suitable with pre-existing weaknesses such as jointed rock and pre-existing fractures. Hydraulic fracturing is also limited by the faulting mechanism. For instance, if the stress field is controlled by reverse faulting, where vertical stress is the minimum principal stress, fractures will be formed in the horizontal plane. In this case, horizontal stress magnitudes or orientations cannot be estimated in this case (Gaines, et al., 2012). Moreover, due to its stimulation on well production and favourable faulting condition, hydraulic fracturing is widely used in petroleum engineering.

Hydraulic tests on pre-existing fractures

HTPF was initially proposed by Cornet and Valette (1984), which was designed to overcome the shortcomings of conventional hydraulic fracturing method. Comparing with the conventional method, HTPF focuses on the re-opening of pre-existing fractures in the sealed section. This technique aims to determine normal stresses acting perpendicular to pre-existing fractures, which equal to the shut in pressure generated by fluid injection. Accordingly, it is important to gather precise locations and orientations of fractures prior to the commencement of the fluid injection. This is usually achieved by borehole imaging techniques, such as Mosnier tool (Cornet, et al., 2003). The sketch is shown below in Figure 2.
Since there are no fractures induced by HTPF, it is less limited to the geological conditions. Comparing with the conventional method, tensile strength and pore pressure of the surrounding rock are not involved in the estimation, so that the measurement of rock properties is not mandatory. With sufficient tests conducted, 3D stress field can be computed using HTPF. However, due to strict requirements of fracture locations and large number of tests, HTPF is more time intensive than the conventional method. In general, 20 successful tests are necessary for a proper 3D stress interpretation (Ljunggren et al., 2003). Thereby, HTPF is based on the assumption that the fracture orientation is persistent. Distorted fractures can lead to overestimation of shut in pressure, which consequently results in inaccurate evaluation of principal stress magnitudes.

OVERCORING

Overcoring is also a widely used stress measurement technique, particularly in mining industries. The estimation is based on the strain deformation within a pilot hole. To overcome different limitations, a range of methods were developed with similar procedures and the same principal, i.e. linear elasticity. Depending on instrumentations and requirements of pilot holes, these methods can be divided into three types:

- Displacement Measurement
- Soft Stress Cells
- Overcoring without Pilot Holes

Displacement measurement

Deformation cells are common instruments used in overcoring. The change in borehole radial displacement is measured during overcoring using six cells, and converted to stress magnitudes in conjunction with rock properties such as Young’s modulus and Poisson’s ratio, which are measured from cored samples. The two prevalent displacement measurement instruments are US Bureau of Mine cell (USBM) and SIGRA IST. In Australia coal industry, the most used instrument is SIGRA IST.

A borehole is initially drilled to the desired measurement depth. By then, a pilot hole is advanced from the bottom of the borehole at the centre using a barrel. At the completion of drilling, equipment is retrieved together with cored rock samples. Cored samples are transported to laboratory for rock properties testings, usually biaxial tests. Afterwards, strain gauge is installed in the pilot hole and overcoring is commenced at the diameter of borehole. Subsequently, the radial displacement recording by the strain gauge is collected and used for stress estimation. A schematic diagram is shown in Figure 3. This technique provides stress measurements at high accuracy and can be applied to various geological conditions. For example, it is not constrained by faulting mechanisms. Besides, strain cells are recoverable which can later be used for multiple tests. This would reduce associated costs. Moreover,
since cables are not connected from cells to the computer, theoretically, it is not limited by depth.

However, there are also a series of shortcomings related to the technique. Similar to other methods, the displacement cell is based on the assumptions of linear elasticity and rock homogeneity and isotropy. Clearly, rock mass doesn’t satisfy these conditions, which infers it has inherent uncertainties as other techniques (Gaines, et al. 2012). Given the effect of water table and the continuity of cored sample under high stress conditions, this method is practically only suitable to shallow depth. Thereby, multiple tests are required to carry out for complete stress field estimation, which can be time consuming.

![Figure 3: Overcoring procedures (Ljunggren, et al., 2003)](image)

**Soft stress cells**

The principal of soft stress cells is to stick highly sensitive strain gauges onto the rock around the borehole using temperature-specific gluing packs, so that gauges can rapidly become a part of the rock. As soon as overcoring is conducted, surrounding rock as well as gauges can experience similar deformation. Later, stress field can be estimated providing data obtained from strain gauges. In general, well-known soft stress cell instruments include CSIR, CSIRO HI Cell and Borre Probe.

Soft stress cells determine the stress field within one borehole measurement only, which is more favourable than displacement measurement. In terms of stress dimensions, soft stress cells offer an accurate 3D stress estimation rather than 2D through a rotational and continue logging, except CSIR. Furthermore, there is a major advantage which distinguishes Borre Probe with other overcoring techniques, which is its application in deep, water filled boreholes. (Gaines, et al., 2012).

In opposition, this technique also has a lot of disadvantages. Unlike USBM or SIGRA IST, soft stress cells are difficult to be recovered. Epoxy based glues is not applicable in humid and dusty environment, including coal mines (Coetzer, 1997). The thickness of glue and its associated temperature effect may influence the accuracy of the measurement. In line with USBM and SIGRA IST, unbroken long cores are required for successful measurements. It is usually difficult to be achieved due to pre-existing fractures, discing or joints. Under high stress conditions at deep locations, this problem is amplified for Borre Probe measurement.

**Overcoring without pilot holes**

Doorstopper is a special overcoring method which doesn’t require a pilot hole. Instead, the borehole bottom has to be carefully polished to ensure it is flat so that the strain gauge can be glued and attached to the rock. It is principally the same as other overcoring techniques which records the deformation of rock induced by overcoring. Atomic Energy of Canada Ltd. modified this instrument for deep measurement, namely, Deep Doorstopper Gauge System (DDGS) (Thompson and Chandler, 2004).
Overcoring methods with pilot holes generally require more than 300 mm overcoring length, whereas doorstopper only needs 50 mm. As a result, doorstopper is less time consuming and can be conducted at deep locations, which can be up to 1000 m. It also provides a higher successful measurement rate in relatively weak or broken rock or even in high stress conditions, due to the un-essentialness of the pilot hole. With modifications, DDGS can also be implemented to deep, water filled boreholes. Therefore, in deep stress measurement, it is more favourable than Borre Probe.

Although the pilot hole is not necessary for doorstopper, polishing and preparation of the borehole bottom are essential. To obtain the stress field, the borehole has to be parallel to the vertical principal stress, indicating that doorstopper only permits 2D stress determination perpendicular to the borehole.

**BOREHOLE BREAKOUT**

Once a hole is drilled underground, the in-situ stress field is disturbed and the compressive stress is concentrated on the rock around the borehole. When the compressive stress overcomes the rock strength, the failure of rock occurs at opposite areas around the borehole wall. Induced void created by rock detachments or flakes is so-called 'borehole breakout', which initiates and propagates along the minimum principal stress direction, see Figure 4. To measure breakouts, logging systems are required to start from the surface of the borehole to the maximum depth. Most frequently used apparatus are borehole televiewer, formation scanner and calliper.

The classical concept of borehole breakout was initially proposed by Bell and Gough (1979), who interpreted the stress field around the borehole based on Kirsch solution. They suggested that the maximum compressive stress concentration is at the minimum principal stress direction, whereas the minimum compressive stress concentration is at the maximum principal stress direction. Thus, the maximum breakout depth should be aligned to the minimum principal stress direction. Based on this concept, borehole breakout has been developed as a reliable measurement technique for stress orientations.

A series of attempts were made to correlate the stress magnitudes to breakout dimensions according to elastic conditions (Zoback, et al., 1985; Barton, et al., 1988; Chang, et al., 2010). It is clear that both breakout width and depth are directly related to stress magnitudes, as stress magnitudes increase, the width and depth also increase. However, due to various reasons, the accuracy of stress field estimation is relatively low. The primary reason for this is the time dependent breakout deepening (Zoback, et al., 1986). Stress re-distribution induced by rock failure and plastic deformation cause the change in compressive stress at breakout tip. As time goes, the rock at breakout tip may fail and advance forward due to accumulated stress concentration. The breakout measurement usually takes place at least after few hours of drilling, which means the depth measured is deeper than initially formed. In this case, the stress calculation based on elastic conditions yields unreliable results. Thereby, pre-existing fractures around the borehole can also result in the elongation of breakout, which further disrupts the estimation. In addition, current field equipment have drawback in measuring breakout depth. When fractures exist at the area of measurement, times for the equipment to receive emitted acoustic waves are considerably disrupted. Since there is no reference, the depth measurement cannot be verified and hence may not be used. Other than that, breakouts have to exist to allow the stress measurement take place and it rarely happens at depth above 100 m.
On the contrary, this method offers some advantages rather than other techniques. Firstly, it is simple and quick method for primitive stress estimation as only geometrical measurement is required. In Australia, every borehole drilled has to be logged. This means breakout data has already there, the estimation can be directly carried out. Secondly, it enables deep stress measurement which normal methods may not reach. Another advantage is that the shape and size of breakout can indicate the high stress concentration zone, which can be treated as a primary tool of coal burst detection and strata control. After the equipment scans existing breakouts within the borehole, stress profile at different depth can be interpreted in one measurement. This is cost effective and time saving.

**FLAT JACK**

Flat jack is another cost effective method conducted at the surface of the excavation. The concept is to calculate the stress based on the pressurisation of a flat jack in a slot. Two points A and B are selected and measured continuously by strain gauges which are followed by a nearby slot cutting. Afterwards, a flat jack is inserted into the slot and pressurised until the distance between A and B is back to the original distance. The pressure at this point, the cancellation pressure, is assumed to be the average normal stress across the slot and subsequently the stress field can be interpreted by multiple tests. This method is simple, cheap and easy to be carried out while the elastic modulus is not required for the calculation. Conversely, flat jack is only applicable at the surface of the excavation where the rock is closely to be overstressed. This may lead to unreliable estimation.

**OTHER MEASUREMENT METHODS**

There are also other stress measurement approaches which cannot be covered in this article, such as borehole slotting, core discing and acoustic emissions. Each method has its major advantages in particular conditions. For example, acoustic emissions are suitable for rock at shallow depth where principal stress magnitudes are lower than rock strength.

**SUMMARY OF METHODS AND FUTURE RESEARCH PLAN**

In previous sections, popular in-situ stress measurement techniques have been reviewed together with their advantages and disadvantages. It is clear that hydraulic fracturing is the most suitable method in normal faulting, whereas overcoring is widely used in mining industries providing reliable measurement. On the contrary, most methods only provide point measurement per test, the complete stress field profile over depth has to be obtained through a series of successful tests. Practically, it is economically unfavourable and time consuming to conduct multiple tests at different depths for ground controls.

Borehole breakout is a natural event which usually occurs different levels at depth below 100 m. With scanning or calliper measuring from surface to bottom of the hole, breakouts at various locations can be recorded in one scanning activities. In Australia, every borehole drilled has to be logged, which means breakout data is available already. Thus, borehole breakout presents a great potential for stress profile estimation over depth. To develop a primitive stress estimation technique based on borehole breakout on ground controls, it is essential to identify influential factors that affect breakout dimensions. To achieve this, a series of experiments have been carried out at China University of Mining and Technology.
(CUMT) via a true triaxial test machine and undergone CT scanning after tests. CT scanning data is still under processing and influential parameters are to be recognised. A thorough parametric study is proposed to be performed using numerical software PFC. It aims to study the relationships between breakout dimensions and each influential parameter. Together with field data collected from mine sites, a primitive stress estimation method using borehole breakout is expected to be developed for ground control purpose.

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THE REDISTRIBUTION OF STRESSES AROUND LONGWALL EXTRACTION PANELS IN BEDDED ROCK MASSES

Ross Seedsman

ABSTRACT: Longwall mining is conducted in bedded rock masses which are not only inhomogeneous but also transversely isotropic. Assuming isotropic rock mass properties can substantially under-estimate the impact longwall extraction can have on the redistribution of horizontal stresses. By invoking the transverse isotropic parameters derived from data on stress-relief roadways it is demonstrated that stress modifications can extend laterally 8 to 10 times the effective excavation height. For longwalls this may mean stress changes could extend up to a 1 km distance. Recognition of the role of transverse isotropy has implications to the understanding of stress concentration effects at the maingate/face corner and also predicting mining conditions in new longwall districts.

INTRODUCTION

In geotechnical engineering the term isotropy refers to a condition whereby the properties are the same in all directions. This differs from the concept of homogeneity which refers to the presence of the same material – the layering of different rock types often seen in numerical models recognises inhomogeneity but not transverse isotropy. Sedimentary rock masses are characterised by the presence of laterally continuous bedding discontinuities at spacings ranging from millimetres to tens of metres and must be considered to be transversely isotropic at all scales. Transverse isotropy in an equivalent continuum invokes the same elastic properties in one plane and a different set of properties out-of-plane and hence can be an analogue to bedding in sedimentary rock masses. Despite this it is common practice to model such rock masses as being isotropic both in terms of their deformation properties and their strength. Seedsman (2011) highlighted the possible role of transverse isotropy in modifying the stress redistribution about coal mine roadways and evidence from coal mines and tunnels was interpreted to propose that a Young’s Modulus/Independent shear modulus (E/G) ratio of 100 could be applied to sedimentary rock masses with bedding spacings in the order of 200 mm.

Recently Gale (2013) and Galvin (2016) have published analyses of stress distributions around longwall panels using the isotropic assumption and both authors determined distortions to the pre-mining stress field extending out only about 30 m from a longwall face. Recent observations of ground conditions in two longwall mines at depths of 300 m and 550 m have suggested that longwall extraction can modify the horizontal stress field to distances in excess of 500 m. This paper examines how the transverse anisotropy parameter proposed for the roadway scale can also be applied to the scale of a longwall extraction goaf.

LITERATURE REVIEW

Based on stress monitoring in a mine at 500 m depth Gale and Mathews (1992) proposed reductions in horizontal stress could be achieved up to 40 m distance from a roadway that had damaged (“softened”) zones extending 4.5 m to 5 m into both the roof and floor. Seedsman (2017) used their case study data to determine that an E/G ratio of 100 could be used for interbedded laminites and sandstones. Compared to the isotropic case the use of this E/G parameter extended the distance for stress relief from 15 m to 60 m (stress ratio reduced from 1.6 to 1.4 in Figure 1). This represents a distance/excavation height aspect ratio of 60/8 or 7.5, which if applied to a longwall void defined by combined caved and fractured zones of 100 m would imply stress changes at 750 m distance. In regard to the vertical stress there are significant differences close to the excavation which may be a numerical limitation, but the width of vertical stress abutment is similar at about 20 m for the isotropic and transversely isotropic cases.

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University of Wollongong, February 2018
In Gale (2013), modifications in the horizontal stress field about 10 m above the mining horizon extended at least 50 m – 100 m ahead of the faceline (Figure 2). The same work presents data on the relative magnitude of the induced to pre-mining major principal horizontal stresses as a function of the angle between the pre-mining principal stress direction and the retreat direction. The published data does not include information on the orientation of the induced stresses or the magnitudes of the minor horizontal stresses so it is not possible to resolve the stresses to those acting across a roadway.

Gale (2013) considered the three-dimensional stress redistributions about longwall panels which were modelled as voids with heights of 200 m; the report references only isotropic elastic parameters and inspection of the included figures suggest that isotropy was indeed assumed in as much as significant stress changes were only recorded within about 30 m of the goaf zone. It is noteworthy that the modelled concentration factors in Figure 3 are less than those that have been measured (Figure 3). There was negligible concentration of horizontal stresses at 75 m distance or at a distance/void height aspect ratio of 0.375 (red lines in Figure 3).
In discussing interaction between workings Galvin (2016) used an isotropic model with the longwall goaf modelled as a 150 m high soft inclusion with no ability to transmit horizontal stress (effectively a void) and with a rigid floor and found that stress changes were contained within about 30 m of the excavation. In a numerical study Suchowerska et al (2013) examined the role of transverse isotropy in modifying the vertical stresses under coal mine pillars with E/G values of 2.5 to 25 and found that the peak vertical stress increased by 42% over this range; no validation data was provided.

MODEL

Longwall extraction void

In this work an isolated longwall goaf is modelled as a 250 m wide, 1000 m long void (Figure 4) and the height of the void is taken as 100 m based on the empirical models for the prediction of the height of the combined caved and fractured zone assuming a 3 m coal seam (Bai and Kendorski, 1995). A caving angle of 70° has been adopted. Based on reports of fracturing in the floor inducing gas inflows from underlying seams, a 40 m thickness of floor failure is included in the modelled void with an examination of this assumption conducted by considering 0 m and 10 m of floor failure.

Figure 4: Modelled shape for longwall void and intersection

The rock mass is modelled as a single Transversely Isotropic (TI) material with a Young’s Modulus/Independent shear modulus ratio (E/G) of 100 and a Poisson’s ratio of 0.2. Isotropic material is also modelled for comparison purposes. The adopted stress regime is major horizontal:minor horizontal:vertical in the ratios of 2:1.5:1.0. In the model the x axis is aligned parallel to the faceline (= direction across the gateroad), the y axis is the direction of face retreat, and the z axis is vertical. Orientation of the stress field with respect to the y axis varied between 0° and 180°. Two and three dimensional codes (Examine2D and RS3) were used; both can be applied in a vertical section at mid-panel but only RS3 can be used for the horizontal section and at the panel ends. Results are presented for the horizontal stress acting across the roadway ($\sigma_{xx}$), horizontal stress parallel to the roadway ($\sigma_{yy}$), the vertical
stress ($\sigma_{zz}$), the ratio of $\sigma_{xx}/\sigma_{zz}$ (referred to as the K ratio), and the ratio of the in-plane major principal horizontal stress to the initial vertical stress.

**INTERSECTIONS**

The geometry of a three-way intersection is shown in Figure 5 and includes 5.2 m wide roadways and an 8.6 m diameter circle in the intersection: the intersection was modelled with a 3 m mining height. The codes RS3 and Examine3D were applied. In the model the major principal horizontal stress is aligned parallel to the through-going roadway. The purpose of these analyses was to examine the increase in the height of failure above an intersection so that plane-strain analyses of roadways can be extrapolated to the more complex three-dimensional case of an intersection. As will be discussed, the visualisation of the RS3 results were impacted to some degree by the meshing. In RS3 a friction angle of 50° was used in combination with the E/G ratio of 100 (Seedsman 2017). For Examine3D to compensate to some degree for the inability to invoke transverse isotropy a friction angle of 35° was selected to give a similar failure height as the RS3 analysis.

**RESULTS**

**Mid panel**

Figure 5 presents results for the concentration of the vertical stress and the change in the ratio of the horizontal to vertical stress along a mid-panel vertical section. The width of the zone of increased vertical stress is less for the TI assumption (<75 m) compared to the isotropic (250 m). The TI assumption gives a wider zone for the reduction in horizontal stress compared to the isotropic. The K ratio for the isotropic assumption approaches far-field value of 1.5 at about 300 m offset whereas at the same distance the K ratio for the TI material is 0.77 (Examine2D) or 0.99 (RS3). In fact the far field stress conditions are not attained within the 800 m that have been modelled for the TI assumption.

The mining significance of this result is that isotropic models substantially underestimate the extent of horizontal stress relief provided by a longwall goaf. Isotropic models would suggest that the driveage for the next panel would be in a stress field similar to the pre-mining condition, whereas the horizontal stress could be about half the pre-mining levels. It is noted that there are negligible differences in the prediction of the vertical stress levels for the driveage in the next panel.

**Floor failure**

Figure 6a presents the results of 2D analyses with three different floor void thicknesses. The extent of floor failure has a major influence on the pattern for the TI material but less of an influence for the isotropic material. Once again the TI material returns a wider zone of impact for the 10 m and 40 m floor voids. For the isotropic assumption and the TI assumption with no floor void the change in the K ratio is confined to a width less than about 200 m.

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![Figure 5: Comparison of isotropic and transversely isotropic assumptions for a vertical section at mid-panel (Examine2D model)](image_url)

Figure 6b shows the sensitivity of
the K ratio developed at 400 m offset from the longwall void to the height above and below the mining level for two values of the floor void thickness. There are negligible differences if the floor is considered rigid or if the rock mass is considered isotropic. The combination of transverse isotropy and floor void thickness results in substantial variations – a 0.1 change in the K ratio for a 10 m change in vertical location.

### Figure 6: Influence of the floor failure zone on the K ratio for a section normal to the longwall void located at mid-panel

The mining significance of this result is that there may be a substantial stress reduction in the travel road behind the longwall face. These may not be sufficient to induce tensile failure but the stress reduction may result in loosening or collapse of the immediate roof if it had undergone failure at the maingate corner.

### Maingate corner

Analyses for the maingate corner utilised RS3. Figure 7 presents data in terms of the resolved stresses and the in-plane major principal stress for the case of the longwall retreating parallel to the major principal horizontal stress. The figure shows a rotation of the direction of the major stress with a concentration similar to that reported attained about 20 m from the face line (about 1.6 in Figure 2). Changes in the magnitude of the horizontal stresses extend out about 150 m and there are some rotational impacts out to 300 m and beyond.

### Figure 7: Details of horizontal stress components for the TI assumption and the roadway parallel to the major principal horizontal stress
Resolved stresses

Figure 8 compares the TI and isotropic assumptions and while the patterns are somewhat similar the magnitudes of the changes are substantially different. For example the modelled vertical stress at the faceline is 2.1 times the pre-mining value for the TI case compared to 1.55 for the isotropic case. The concentration of the $\sigma_{xx}$ horizontal stress at the faceline is also greater at 2.75 compared to 1.75. An important observation is that close to the faceline the K ratio decreases as a result of a greater concentration of the vertical stress compared to the horizontal stress.

Figure 8: Horizontal and vertical stress acting in the belt road (20° orientation to the major principal horizontal stress)

The concentration of the magnitude of $\sigma_{xx}$ compared to the pre-mining value as a function of the orientation of the roadway with respect to the direction major principal horizontal stress (Figure 9) for the TI case is much greater than that reported by Gale (2013) as presented in Figure 3b. As expected the concentration levels are greater for the TI case and are somewhat similar to the measured data in Figure 2 if the frame of reference is taken at about 12 m from the face line. The mining significance of this result is that the measured stress concentrations are better explained by a transversely isotropic model compared with an isotropic model.

Figure 9: Concentration of $\sigma_{xx}$ as a function of the retreat direction and distance from the face line

Stress shadows

Figure 10 shows the relative magnitude of the $\sigma_{xx}$ stresses developed on a horizontal plane at the seam level when the direction of longwall retreat is 20° from the direction of the major horizontal stress: the approximate location for a K ratio of 1.0 highlighted with white dashes.
In Figure 11 the in-plane stresses are shown for a line located at the mining level and 250 m offset from the longwall. There are significant reductions in $\sigma_{xx}$ to a K ratio of about 1.0 inside of about 250 m from each end of the void. There are minor variations in $\sigma_{yy}$.

Figure 10: (a) Relative magnitude of horizontal stress acting across the gateroads ($\sigma_{xx}$) for an alignment of 20$^\circ$, (b) section drawn at 250 m off set.

The significance of this result is that when a longwall face is located adjacent to a previous goaf there will be lower horizontal stress magnitudes at the face/gate corner so long as the face/gate corner is inside a line drawn at about 45$^\circ$ from the previous panel ends. Perhaps it would be better to refer to a stress shadow beside a longwall instead of a stress notch at the ends.

Intersections

The isosurfaces for a strength factor of 1.0 (Figure 11) show an increase in the height of failure above an intersection compared to the roadway. The ratios of the failure heights are similar to the ratio of the inscribed diameter to the roadway width (Table 1).

Figure 11: Examine3D and RS3 models showing failure heights above intersections

Table 1: Height of failure above roadways and intersections

<table>
<thead>
<tr>
<th></th>
<th>Roadway (m)</th>
<th>Intersection (m)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Examine3D</td>
<td>2.8</td>
<td>4.74</td>
<td>1.66</td>
</tr>
<tr>
<td>RS3</td>
<td>3</td>
<td>5.04</td>
<td>1.68</td>
</tr>
<tr>
<td>Diameter ratio</td>
<td></td>
<td></td>
<td>1.65</td>
</tr>
</tbody>
</table>

The significance of this result is that a correction factor based on the ratio of the diameter of inscribed circles can be used to extend plane strain (2D) models of failure heights above headings to failure heights above intersections.

DISCUSSION

This paper has shown how substantially different understandings of the stress fields around longwalls may be obtained depending on the selection of input parameters to simple
numerical models. Using a ratio of the Young’s modulus to Independent Shear Modulus of 100 provides a good match to measured stress relief around roadways and extending this parameter to a longwall goaf indicates the possibility of substantial stress changes extending almost 1 km.

**Applicability of the E/G ratio**

Brady and Brown (1985) provide an equation to estimate the independent shear modulus for a rock mass with a single set of parallel joints (bedding) which requires an estimate of the joint shear stiffness. Bandis et al (1983) published an empirically-derived relationship that involves the normal stress, the basic friction angle, the Joint Roughness Coefficient, the Joint Compressive Strength (equal to UCS for a clean joint), and the length of the joint. Referring to Figure 12, an E/G ratio of 100 is consistent with an average bedding spacing of 150 mm for mining depths of 400 m to 500 m. It may be possible to use this relationship to estimate alternative E/G ratios for different bedding thicknesses and hence to extend the analyses outlined in this paper to more thickly bedded units.

**Figure 12: Variation in the E/G ratio as a function of bedding spacing (ϕ_r=30°, JRC=1, JCS=UCS=60 MPa, L=0.5 m, modular ratio=250)**

**Mining induced changes to the virgin stress field**

Based on this work it is suggested that only the first panel in a new mining area, or when subsequent panels are longer, will the gateroads be exposed to the virgin stress field. This means that contiguous longwalls operate in a much reduced horizontal stress environment except near the start or end lines of the previous longwall. The first longwall panel in a new mining district may be exposed to higher horizontal stresses even if the depths of cover and the structural geology are similar. There are a number of implications to this. Firstly, the design and interpretation of stress measurement programs need to consider the distance from existing longwall extraction. Secondly, the stress footprint of a longwall may be much wider than previously considered and this may need to be considered when considering so-called far-field subsidence movements.

**Brittle failure near the maingate corner**

The presentation of the data in terms of the K ratio and the relative change in the vertical stress allows ready integration with the TIB brittle failure criterion (Seedsman, 2017). The rock strength index (RSI) can be calculated from the Unconfined Compressive Strength (UCS) and the vertical stress estimated from the depth of cover and the vertical stress: RSI = UCS/(Depth * average overburden density) so that for a UCS of 50 MPa, a mining depth of 350 m and a density of 2500 kg/m³ the pre-mining RSI is 5.7.

Figure 13 presents the stress path for an isolated longwall panel in a stress field with principal stresses in the ratio of major horizontal:minor horizontal:vertical stress of 2:1.5:1. Three stress paths are indicated – parallel, 45° and normal to the major principal horizontal stress. As discussed above the impact of the longwall extraction begins about 300 m distant and the vertical stress starts to increase at about 100 m distant. The stress condition associated with
the greatest height of TIB failure develops about 24 m from the faceline and the failure heights for the maingate are 5.0 m, 6.6 m and 8.2 m for the three respective orientations and hence 8.3 m, 11 m and 13.5 m for the intersections.

Figure 13: Longwall stress paths projected onto TIB design chart (bold numbers indicate distance from faceline, diagonal and vertical lines are heights of brittle failure)

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STRONG WEIGHTING EVENTS IN SHALLOW MULTI-SEAM LONGWALL MINING

Weibing Zhu¹, Jialin Xu², Jinfeng Ju³, Qingdong Qu⁴

ABSTRACT: When longwalls are operated underneath previously mined-out workings in shallow coal seams in the Shendong Coalfield, China, extremely strong weighting events often occur and sometimes result in the longwall supports becoming ironbound. The occurrence of such events was analysed through characterising the movement of the rock structures formed by the excavation of the upper seam as well as the associated load transfer mechanism. Three typical mining conditions that have caused frequent strong weighting events were identified: (1) when the longwall face is mining underneath the uphill section of a valley terrain; (2) when the longwall face is advancing out of upper coal pillars; and (3) when the longwall face is mining underneath upper chamber coal pillars. The mechanisms caused strong weighting events under each condition and corresponding prevention and management strategies are discussed.

INTRODUCTION

Shallow longwall mining can be regarded as having relatively a low intensity of weighting events in comparison to mining at depths. However, coal mines in the shallow Shendong Coalfield of China have experienced frequent strong weighting events and associated excessive convergence on longwall supports which sometimes resulted in the longwall becoming ironbound, a state of roof supports being fully compressed. Surface cracks with associated scarps w often accompanied such events.

Shendong coalfield in the northern part of China is a typical mining district that produces coals at shallow depth of cover. In recent years, the coalfield has become one of the largest coalfields in the world, producing hundreds of million tons of coal per annum. With the topmost minable seam being mined, some coalmines in the coalfield have extended further to excavate lower seams under previously mined-out workings. Practices so far have showed that the load created on the longwall supports is greater than when mining the upper seam. Sometimes extremely strong weighting events with rapid convergence on longwall supports have occurred. Even with longwall support load capacities as high as 18,000 kN (an intensity of 1.52 MPa) in some coalmines, strong weighting events have still occurred.

The load imposed onto the longwall supports is closely associated with the pattern of overburden strata movement in particular the cantilevered blocks of roof strata hanging over the longwall face (Qian, et al., 2010). Generally, when the longwall face advances, the main roof will break at a certain interval, resulting in periodic weighting onto the longwall face. In a single seam-mining scenario, the additional load generated during periodic weighting is mainly associated with the main roof and the overlying strata that breaks together with it. However, such a mechanism is not sufficient to explain the excessive loading events that are being experienced in mining the lower coal seams in the Shendong Coalfield.

The concept of Key Stratum (KS), which is characterised as a strong layer that controls the movement of the strata between it and the next KS above, has been increasingly adopted in China in analysing the process of overburden strata movement as well as longwall weighting events. In the last decade, it has been gradually recognised that the breakage of the second KS from the working seam (the 1st is the main roof) can have a significant effect on the support load (Xu, et al., 2009, 2014). Based on the KS concept, the occurrence of strong weighting events in the Shendong Coalfield was analysed. Three typical mining conditions

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were identified that can result in extremely strong weighting and sometimes an iron bound longwall face (Zhu, et al., 2010, 2017 and Ju, et al., 2015).

These conditions are:

1. When the longwall face is mining underneath the uphill section of a valley terrain Second point,
2. When the longwall face is advancing out of an upper coal pillar,
3. When the longwall face is mining underneath an upper chamber coal pillar.

This paper presents the three typical conditions as well as the mechanisms that cause the events. Corresponding management strategies were also briefly discussed in the paper.

**TYPICAL CONDITION 1**

A typical case

Valley terrain with a height of 30~70 m is commonly present in the Shendong Coalfield. The geological structure is formed due to flood erosion. In the lower part of the valley, bedrocks are often seen underlying the top loose soil bed.

Daliuta mine is one of the modern coalmines located in the Shendong Coalfield. Abnormal strong weighting events have occurred several times in the mine when the longwall face was advancing underneath the valley terrain. Figure 1 shows the geological conditions along the LW12304 panel, which extracted the 1\textsuperscript{1} coal seam. The upper 1\textsuperscript{2} seam, 6~27 m above the mining seam, was previously mined out. The depth of cover of the 1\textsuperscript{2} seam is 40~120 m. When the lower 12304 face was advancing underneath the valley terrain, excessive loading of the longwall supports which resulted in rapid convergence of the longwall supports occurred twice at the locations shown in Figure 1.

![Figure 1: Geological conditions of LW12304 at Daliuta mine](image)

The first event occurred when the longwall face was undermining the middle of the terrain slope, as shown in Figure 1. Consequently, the coalface between supports No.36 and 76 experienced significant spalling with a depth of 1~2 m, and the roof caved about 1.5~2 m. Several supports rapidly converged 200 mm with the yield valves opened completely. On the ground surface, cracks parallel to the longwall face were observed across the valley with a width up to 1.2 m and associated scarps with a height greater than 1.0 m. Figure 2 shows the surface cracks on the slope of the valley.

The second event occurred when the longwall face was undermining the plateau of the terrain. As a consequence, roof falls were observed between supports No.46 and 66, and rapid convergence of 420 mm was observed at the longwall supports. Surface cracks were created with a width up to 4~5 m and associated scarps of 1~2 m high.
Figure 2: Surface cracks with scarps on the slope of the valley

Postulated Failure Mechanism

According to the geological conditions, two KSs were identified in the overburden. The primary KS (PKS), the topmost KS that controls the movement of the entire strata from the layer up to the ground surface, was found missing within the valley area, as shown in Figure 3. In such a condition, the lateral force, which would have served as resistance to the movement of the PKS blocks that were broken during the excavation of the upper 1/2 seam, was missing. This would lead to the blocks not being able to maintain stability or provide self-support during extraction of the lower seam. Thus the weight of the PKS and its overlying strata would be transferred down to the main roof of the lower seam and then onto the longwall shields, as shown in Figure 3. As a result, rapid and excessive roof convergence would be inevitable. Meanwhile, large cracks on the ground surface along with associated scarps were created.

Figure 3: Movement of rock blocks when the coalface was beneath the valley slope

Management

Based on the mechanism discussed above, the risk of abnormal weighting events can be assessed through identifying whether the PKS is present throughout the valley areas. In addition, the risk is also highly associated with the height of the strata overlying the PKS. The detailed stratigraphic information and relevant strata mechanical properties are therefore critical to the risk assessment and management. Clear signs should be placed in
underground roadways at corresponding locations that defines the risk zone. In addition, increasing support capacity is an effective and direct control measure. The capacity of the support must be determined in accordance with the detailed stratigraphic condition in the risk zone to ensure that it is sufficient to sustain the load transferred from the PKS and its overlying strata. Taking the LW12304 as an example, the support load capacity should be increased to 13,867 kN based on the stratigraphy present at the strong weighting event sites.

**TYPICAL CONDITION 2**

**A typical case**

A number of strong weighting events with rapid convergence have been experienced while the longwall face was advancing out of the coal pillars in the upper mined-out seam. Since 2007, these types of events have occurred about ten times. One such event occurred at LW12304 of Daliuta mine. Figure 4 shows the relative location of LW12304 and the upper coal pillars. A sharp increase of support resistance was observed between supports No.63 and 105 when the longwall face was about 3.4 m outbye the upper coal pillar. Consequently, the yield valves were activated. The maximum convergence of the supports reached 1200 mm and some of the supports became iron bound. The face spalling was about 1.1 m deep into the coal and the roof caved about 1.2 m high. This event stopped coal production for two days.

![Figure 4: The relative location of LW12304 and upper inclined coal pillars](image)

**Postulated Failure Mechanism**

Figure 5 shows the structure formed by blocks of roof strata when a coal pillar of the upper seam was undermined. It is clear that the structures of the KS blocks formed during excavation of the upper seam were different between the pillar and goaf zones. Due to the existence of the coal pillars, the KS block over the pillar zone formed a three-hinged structure, as shown in Figure 5. When the lower seam longwall was about to advance out of the coal pillar zone, two blocks C and D rotated towards the coal face along with the subsiding strata below. This rotation caused a repositioning of the middle hinge of the structure (between blocks C and D) thus lowering it relative to the other two, breaching the stability of the three-hinged blocks. In such a situation, the load previously taken by the three-hinged structure would be transferred down to the longwall supports, leading to excessive roof convergence and longwall supports becoming iron bound.
Management

To minimize strong weighting events in such a scenario, it is critical to prevent the excessive rotation of the three-hinged blocks that were formed over the coal pillars. In addition to increasing the load capacity of the support, it is possible to take measures to reduce the rotation of the upper blocks. Methods that can be applied include blasting the edge of the upper coal pillars to stimulate the rotation far ahead of the longwall face, and backfilling the goaf area near the pillar zone to prevent excessive rotation of the rock blocks. Moreover, the situation of the longwall loading environment beneath the upper pillar zone needs to be taken into consideration as early as in the planning and design phases of the lower seam excavation. The alignment of longwalls at the two mining horizons can be optimised so as to minimize excessive loading events.

TYPICAL CONDITION 3

A typical case

LW31201 of Shigetai coalmine in the Shendong Coalfield (Figure 6) extracted coal from the 3-1 seam. The longwall was 311.4 m wide and 1,865 m long. The mining seam was 3.0~4.4 m thick and the overlying bedrock was about 48~120 m thick. The depth of cover within the longwall vicinity was 110~140 m. The load capacity of the longwall supports was 18,000 kN. Part of the longwall was overlain by previously mined out chambers in the upper 2-2 seam and remnant coal pillars. The distance between the two seams was 30~41.8 m.

Severe weighting events with rapid roof convergence occurred nine (9) times during the operation of LW31201. The most severe one was on 16 December 2013 after the shearer finished its first cut of the day. The shield supports between No. 22 and 135 converged rapidly from 1.3~1.5 m to 0~0.2 m in about 20 seconds, resulting in the face becoming iron bound. A number of leg supports were crushed and the sealing rings were broken. It took 60 days to repair the damaged supports and recover the longwall face, leading to significant economic loss.

Figure 6: Layout of the longwall 31201 and upper coal seam chambers
Postulated Failure Mechanism

Figure 7 shows the likely structure formed by the blocks of roof strata. When there are chamber pillars in the upper seam, the front abutment load ahead of the longwall face will be concentrated onto these coal pillars. If the coal pillars were strong enough to sustain the abutment load, the overburden movement and the load transfer would be the same as that in other zones of the longwall.

However, if the chamber coal pillars failed due to abutment loading, the load transfer would be different. For example, if the coal pillars 1 to 3, as shown in Figure 7 failed, the upper level KS (highlighted in blue) would possibly break into blocks C and E with the broken point located directly above the longwall face. Different from a normal rotation of a cantilevered block, block C would rotate in the reverse direction towards the longwall face. In such a situation, the majority of the load carried by block C would transfer to the longwall face instead of the caved material in the deep goaf. This additional load would result in rapid and excessive longwall support convergence.

Management

The load capacity of the supports used at LW 31201 was 18,000 kN with an intensity of 1.52 MPa. This capacity is already very high leaving limited room to increase the load capacity. Therefore, other measures must be developed.

Based on the mechanism discussed above, the key factor is the rotation of the KS blocks induced by the failure of the upper chamber coal pillars. Therefore, measures that aim to change the patterns of the KS block rotation can be taken. Possible methods include blasting the chamber coal pillars ahead of the longwall face through surface boreholes to make the coal pillars fail prior to being positioned in the abutment zone. This would move the abutment load further ahead of the coal pillar zone and therefore change the KS breaking pattern and rotation levels. Cutting height could also be increased to increase the stroke of legs of the longwall supports to allow more room for convergence.

CONCLUSIONS

Strong weighting events were experienced when longwalls were operated underneath previously mined-out workings in shallow coal seams in Shendong and some other coalfields in China. This sometimes resulted in longwall faces becoming ironbound, significantly threatening miners’ safety and hindering coal production.

Three typical conditions that can induce strong weighting events were identified from the analysis of the structures of the rock blocks and their movement above both horizons of mining. This characterisation has helped coalmines in the Shendong coalfield assess the risks of strong weighting events for subsequent longwalls. These conditions were:
• When the longwall face was advancing beneath the uphill section of valley terrains,
• When the longwall face was undermining upper inclined coal pillars,
• When the longwall face was undermining upper chamber coal pillars.

The mechanisms of these strong weighting events under various mining conditions were analysed as associated with the movement patterns of the rock blocks that were formed during the excavation of the first seam. It was found that, under the three conditions, it was difficult to keep these blocks maintaining stability or self-supporting during the excavation of the lower seam. Once failed, the load previously sustained by the blocks would be transferred to the lower longwall face, resulting in strong weighting events and excessive convergence causing longwall supports to become iron bound.

Prevention and management measures were suggested based on the postulated mechanisms of the various conditions. In addition to the direct control measure of increasing the load capacity of the shield supports, other practical measures that can change the movement pattern of the rock blocks created by the excavation of the upper seam were suggested.

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INTERPRETATION OF ROCK MASS BEHAVIOUR VIA "MULTIPLE GRAPH" APPROACH: ADIT P-CP9 OF THE ALBORZ TUNNEL

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\textbf{ABSTRACT}: The current paper focuses on the application and advantages of the “multiple graph” approach for interpretation of surrounding rock mass behaviour in underground structures. Behaviour of the Argillitic rock mass surrounding Adit P-CP9 of the Alborz Tunnel was interpreted via the “multiple graph” approach resulting in interestingly accurate prediction. The accuracy of the estimation was later observed in the excavation process and afterwards. The observed results are presented which verifies that the “multiple graph” approach can cope satisfactorily with various geological conditions.

\section*{INTRODUCTION}

In construction of underground structures such as tunneling, rock mass classification techniques have been utilized for many years since Terzaghi’s (1946) descriptive methodology or Lauffer’s (1958) proposal on rock mass quality which controls the stand-up time of an unsupported tunnel span. Other systems such as Rock Quality Designation (RQD) by Deere et al. (1967) were also introduced. Yet, Rock Structure Rating (RSR) was the first system for classifying rock mass (Wickham et al. 1972). Pacher et al. (1974) extended Lauffer’s proposal for development of New Austrian Tunneling Method (NATM). However, nowadays the massively used classification systems include Bieniawski’s (1973 and 1989) Rock Mass Rating (RMR) system along with the Q-system which was developed by Barton et al. (1974).

Palmstrom (1995) introduced Rock Mass Index (RMI) for the purpose of calculation of rock mass strength as a construction material. The Geological Strength Index (GSI) was proposed by Hoek and Brown (1997) for both weak and strong rock mass types. Later, Marinos and Hoek (2000) developed a chart in order to make the classification of rock mass by visual inspection much easier. Most recently, Marinos (2014) classified flysch rocks of Northern Greece into 11 groups using GSI.

As a result of theoretical study, an intrinsic characteristic of rock mass namely “rock bolting capability of rock mass or Rock bolt Supporting Factor (RSF)” was introduced by Mohammadi et al. (2017) which can be used for calculation of rock bolting efficiency in a given rock mass. Based on the theory, a mathematical definition of rock bolting mechanism was developed. An application of RSF for coping with the discrepancies of the RMR system in rock mass consisting of interbedding of strong and weak rock layers has been introduced by Mohammadi and Hossaini (2017). Some other discrepancies of the RMR system were reported by Gonbadi et al. (2009).

Russo (2008) proposed a “Multiple Graph” approach to be applied for both preliminary assessment of excavation behaviour and selection of support type at the tunnel face. This system is going to be discussed and used for explanation of geomechanical behaviour of argillitic rock mass of the Alborz Tunnel in Iran where a comparison was made between the RMR system and “Multiple Graph” approach.

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The GDE multiple graph

As a useful tool for both preliminary assessment of the structure behaviour in rock tunneling and selection of support type, the Geodata Engineering (GDE) multiple graph approach has been experienced in many cases (Russo 2008, Russo 2014, Kontrec and Constandinidis 2013, Decman et al. 2013). The graph is composed of four main sections as shown in Figure 1. The properties estimated by the “multiple graph” are presented in Table 1. The first graph is located in the lower right quadrant and the progress is clockwise.

![Figure 1: The GDE multiple graph for preliminary assessment of excavation behaviour (Russo, 2014).](image)

<table>
<thead>
<tr>
<th>Graph 1</th>
<th>Rock block volume + Joint conditions = Rock mass fabric</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graph 2</td>
<td>Rock mass fabric + Strength of intact rock = Rock mass strength</td>
</tr>
<tr>
<td>Graph 3</td>
<td>Rock mass strength + In situ stress = Competency</td>
</tr>
<tr>
<td>Graph 4</td>
<td>Competency + Self-supporting capacity = Excavation behaviour</td>
</tr>
</tbody>
</table>

The rock mass fabric (GSI) which can be a scalar function of two components namely rock structure and joint condition, is estimated through graph 1 (lower right quadrant in Figure 1). Then the rock mass fabric (GSI) as well as intact rock strength (σc) is the base for estimation of rock mass strength (σcm). The next step is to estimate the rock mass competency or competency index (IC) which is defined as the ratio between rock mass strength and tangential stress (σө) on the excavation contour, using rock mass strength (σcm) and in situ stress (σө). However, a simplified assumption here has been adopted by considering the ratio
of in situ horizontal and vertical principal stresses (k) to be equal to 1. The value of IC=1 separates the behavioral response of the rock mass into elastic and plastic domains. Finally, the excavation behaviour based on Rock mass competency (IC) and self-supporting capacity (RMR) is estimated via graph IV (upper right quadrant of Figure 1). In the cases that GSI is already estimated (Russo, 2014).

The “multiple graph” system gives a prediction of surrounding rock mass behaviour which is going to be applied to understand the behavioral aspects of surrounding Argillitic rock mass of Pedestrian Cross Passage P-CP9 in the Alborz Tunnel of Iran.

**CASE STUDY**

The longest ones in the Tehran-North (Shomal) Freeway (TSF) which is the biggest ongoing civil project in the country. The Alborz Tunnels include two main tunnels known as Western and Eastern Tubes as well as an exploratory (Service) tunnel in the middle of the two main ones. There are some adits known as Cross Passages connecting the two tunnels together as well as the two tunnels to the exploratory tunnel. The Cross Passages connecting the main tunnels together are known as Vehicular Cross Passages (V-CP) and the ones connecting the main tunnels to the exploratory tunnel are known as Pedestrian Cross Passages (P-CP).

A schematic view of the Alborz Tunneling Complex is presented in Figure 2. The excavation of the exploratory tunnel has been completed with a TBM with a diameter of 5.2 m. The excavation of Main Eastern Tube is ongoing with the drill and blast method. All the Cross Passages were excavated during the excavation of Eastern Tube which is in its final stages.

![Figure 2: A schematic view of the Alborz Tunnel Complex (Technical Report, 2009)](image)

The surrounding rock mass in the excavation of P-CP9 consisted of tectonised Argillites with low UCS values and a plethora of discontinuities including faults and joint sets. The behaviour of these argillites was checked and properly predicted by the use of “multiple graph” especially as the UCS was low and the properties of rock mass was mainly controlled by intact rock properties rather than the properties of discontinuities. The classification and prediction of behaviour and real rock mass behaviour after excavation are discussed in the next section.

**ROCK MASS CLASSIFICATION IN P-CP9**

The P-CP9 has been excavated through tectonised argillites with a diameter of 6 m. The main features of the surrounding rock mass are presented in Table 2. Based on the RMR system, the surrounding rock mass belongs to Class IV or Poor Rock (RMR: 21-40) where a systematic rock bolt installation along with wire mesh and light to medium steel ribs with spacing of 1.5 m would be enough for supporting the rock. The same has been applied. However, there were many problems in the stability of the structure. Therefore, the aforementioned “multiple graph” system was used for interpretation of rock mass behavior. The process of rock behaviour determination based on the main parameters of surrounding rock mass is shown in Figure 1 with red arrows. As the GSI of surrounding rock mass was obtained directly through rock mass, there was no need for the first quadrant of “multiple graph” to be compiled. Based on the obtained results, as is apparent in Figure 1, the “multiple
graph" predicts a severe squeezing behaviour for the surrounding rock mass which was accurate based on the observed behaviour of the rock mass.

Table 1: Main parameters of surrounding rock mass in P-CP9

<table>
<thead>
<tr>
<th>UCS (MPa)</th>
<th>GSI</th>
<th>Overburden (m)</th>
<th>RMR</th>
</tr>
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<tbody>
<tr>
<td>5-25</td>
<td>41</td>
<td>400</td>
<td>33</td>
</tr>
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The squeezing behaviour of rock mass in P-CP9 was observed before the installation of steel ribs. Even after the installation of steel ribs the severe squeezing continued. The squeezing of surrounding rock mass before and after the installation of the steel ribs is shown in Figures 3 and 4 respectively. In Figure 3, the bending of rock bolt plate is evident indicating that the rock bolt is activated properly, however, the squeezing in this Figure is discernable. Even after the installation of steel ribs, the severe squeezing continued up until it caused the yielding of steel ribs as shown in Figure 4.

Figure 3: Squeezing of surrounding rock mass before installation of steel ribs.

Figure 4: Squeezing of surrounding rock mass after installation of steel ribs.

The facts recorded in this investigation suggest that the “multiple graph” approach can properly predict the behaviour of surrounding rock mass in underground structures. In the case of P-CP9 in the Alborz Tunnel, when the compressive strength of rock material is rather lower than usual, the obvious squeezing of surrounding rock mass was properly estimated whereas the RMR system does not predict the squeezing possibility.
CONCLUSIONS

The “multiple graph” approach was introduced and used to interpret the behaviour of surrounding rock mass in P-CP9 of the Alborz Tunnel in Iran. After a brief explanation on how to work with the “multiple graph” system, the interpretation of rock mass behaviour was carried out. The method predicted the severe squeezing condition in surrounding rock mass of P-CP9 in the Alborz Tunnel which later was verified by actual observations performed on site where the rock bolt plates bent severely and the steel ribs yielded showing the serious squeezing condition in the surrounding rock mass. The “multiple graph” approach is recommended to be used for interpretation of rock mass behaviour as it copes with the diverse geomechanical condition.

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IMPROVEMENTS IN LONGWALL TECHNOLOGY AND PERFORMANCE IN KUZBASS MINES OF SUEK

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ABSTRACT: SUEK operates ten longwalls in its Kuzbass mines. The area is highly gassy, with low gas permeability that effectively precludes pre-drainage, it has gradients from 5 to 25 degrees, weak roof and floor, and can suffer massive inflows of water into the longwall areas. SUEK has been steadily developing technological solutions, improving equipment, extending face lengths and developing longer panels over the last five years. This is now paying off with extremely high levels of output. In May 2017 one face produced 1.407 Mt saleable, and in July 2017 the same face produced 1.567 Mt. However, producing great volumes of coal is only half the problem. The mines are mostly located 5000-6000 km from the ocean, so transport is a major cost. Access to ports in Russia is also limited. SUEK has been forced to develop holistic solutions through the entire value chain in order to become the leading coal mining company in Russia, and to maintain high levels of output from gassy mines.

INTRODUCTION

Siberian Coal Energy Company (SUEK) produces approximately 105 Mt of coal per year. Most of this is export-quality bituminous thermal coal from large underground and opencast mines, plus a small number of coking and semi-soft coking mines. The company also produces sub-bituminous coal from large opencasts. This is sold unwashed for local markets. All bituminous (hard) coal is washed (if necessary) and exported, however several of the Kuzbass operations produce export quality coal without washing, from seams with 7% to 10% ash of high CV coal. The company operates 15 opencasts, with outputs ranging from 3 to 20 Mt/yr, and 12 underground mines with outputs of 3 to 7 Mt/yr, along with 9 washeries. All washeries have been modernised and expanded in recent years so they now wash all ROM coal. In the past most Russian thermal coal mines washed only coarse coal, then blended this with unwashed fines and ultra-fines. SUEK’s new washeries have been constructed using Australian designs, and all older plants that have been completely renovated and re-engineered to wash all coal sizes and to make a closed cycle, eliminating setting ponds. Quality of products is consistent and high.

SUEK exported approximately 52 Mt of low ash, low sulphur, high quality thermal coal in 2016. This comprised 32.1 Mt to Pacific users and 19.8 Mt to the Atlantic markets, and sold 51 Mt of brown coal (sub-bituminous), lower quality hard coal and washery middlings within Russia in 2016.

SUEK moves approximately 80 Mt of coal per year on the Russian Railways system, which is approximately 24% of all the total coal traffic in Russia. In order to get coal onto the rail network SUEK operates 190 locomotives, has constructed and operates 16 loading points and train assembly yards and operates 746 km of track. On average more than 48,000 railcars are dispatched from SUEK mines per month. Russian cars are 69-77 t capacity. Larger railcars are not feasible because of the huge numbers of bridges and tunnels on the main rail routes.

Coal is moved substantial distances to reach ports on the east and northwest coasts. The greatest volume of export coal is mined in Kemerovo province, in the Kuzbass coalfield. These mines are approximately 4800 km from the west coast ports and 5500 to 6000 km to the eastern ports, so rail transport is a major aspect of the business.

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Russia has very few ports that are suitable for export of coal. Established ports were limited to the Black Sea, Baltic, Barents Sea and the Pacific coast near Vladivostok, and this affects the available capacity and the port charges for coal exporters. In order to reduce these constraints SUEK has built its own coal loading port at Vanino on the east coast, and has acquired a controlling interest in Murmansk Commercial Seaport in the northwest. In 2016 we shipped 19.5 Mt through Vanino and 14.2 Mt through Murmansk. Vanino is a new port with ample space for stockpiles and it is equipped with stacker-reclaimers and ship loaders and its capacity is being increased to 24 Mt/yr. Murmansk is constrained for stockpile areas so it loads ships directly from railcars using cranes but its capacity is also being increased to 16 Mt/yr, and it has been deepened and equipped to handle Capesize vessels with rapid turn-around. All ports operate year-round, although Vanino is an ice port.

SUEK has a sophisticated marketing operation with sales offices in Russia and seven other countries, and more than 80% of sales are direct to end users. All coal is quality assured, with SUEK and independent contractors (SGS) sampling and analysing every wagon at mines and also at ports.

The strategy of the company is to mine high volumes from world class deposits, to sell high quality, finished products FOB or FCA to maximise added value and economies of scale, to control the rail transport and shipment of coal and, as far as possible, to sell direct to the end users.

In order to increase volumes mines and opencasts have been equipped with the best available equipment and with high capacity infrastructure for transport of coal, men and materials, gas drainage, ventilation, monitoring and safety.

SUEK is a major player in Russian industry, so it also has a major role in the communities and environment in mining areas. The company invested $15 M in community projects in 2016. Russian environmental law is similar to that of Australia and USA, and SUEK reaches or exceeds all standards for discharges, water quality and dust emissions.

SUEK operates a total of 27 mines and opencasts. These are located in the Kuzbass region in Siberia and the eastern parts of Russia, as shown in Figure 1. These are located along the main railway lines that cross Russia – the Trans-Siberian and the Baikal-Amur Main line (BAM). The figure also shows the main coal ports in Russia, including the three ports controlled by SUEK.

![Figure 1: Location of SUEK mines, ports and Russian railways – Red = SUEK operational areas](image)

The success of the company is highly dependent on maintaining a skilled and productive workforce. All coalmines in Russia are commercially operated and privately owned, so remuneration is competitive. Miners have decent standards of living and stable employment. SUEK has not suffered any layoffs or mine closures as a result of tightened markets, but has continued to develop, modernise and improve standards. Miners in SUEK mines work with the latest and most powerful mining equipment that is available. Most of the face equipment is imported from internationally respected suppliers, such as Komatsu Mining (Joy), Caterpillar (DBT), Sandvik and Famur. Safety standards are extremely good, and strictly regulated. For
example, all underground electrical equipment from the portals to the faces must be explosion proof or intrinsically safe, welding is not permitted underground, all major installations, including transferred longwalls must obtain State approval after commissioning, and mines are inspected almost daily by State inspectors.

In recent years SUEK has started to see the benefits from a period of sustained re-engineering and high investment, combined with technical innovation and sustained development of the workforce. This has culminated with record levels of production from two of the Kuzbass longwall mines:

- In 2015 Taldinskaya Zapadnaya No.1 mine produced over one million tonnes in one calendar month, breaking the previous monthly record for a Russian mine of 797,000 t.
- In May 2017 Yalevskovo Mine produced 1.407 Mt from the longwall in one calendar month. In July 2017 the same longwall produced 1.567 Mt.

In all cases coal quality was maintained within export specification by careful management of the cutting horizon and excellent roof control, so all the coal mined in these two months was sold for export. SUEK has not been able to identify any other instances of such high levels of production in any other country.

SUEK management recognises that monthly or annual records are not normal objectives of modern mining companies. However, there is a tradition of such records in Russia, and they are still a powerful way of motivating personnel. Russia was in a state of economic chaos only 15 years ago, and the changes made in mining are paralleled. Mines have transitioned from State owned, out of date and under-funded enterprises to advanced, modern, completely commercial and profitable businesses. In this environment Stakhanovite records still play a part – they still demonstrate that higher levels of technology can greatly increase production, and the earnings of the workforce. They are not sustainable and must be closely supervised to ensure safety and quality are maintained, but they do serve to drag up average performance in all the mines in the company. As a result, acceptable levels of output from similar longwalls have increased from 400,000 to 600,000 t in a month in recent years.

This paper focuses on the record-breaking longwall at Yalevskovo Mine and the strategies and actions taken by SUEK that have converted the company’s Kuzbass assets from relics of the Soviet era to modern, highly productive, safe and extremely competitive mines – in a remarkably short time.

**KUZBASS MINES**

**Mining conditions**

Much of SUEK’s export coal is sourced from underground mines in the Kuzbass region of Siberia, so these have been a major focus in increasing output and improving productivity. SUEK operates nine underground mines, two opencasts and four washeries in Kuzbass, all mining hard coal (bituminous thermal coal, and high volatile semi-soft coking coal) with a total of 37.7 Mt ROM per year.

The underground mines extract two categories of seams - from 1.6 to 2.6 m thick, and 3.8 to 5.2 m thick. All mines contain multiple workable seams – typically from 6 to 15 seams per mine. Seams are normally worked in descending order, although alternatives have been used for particular reasons.

Seams are relatively free of minor faults, and there are no volcanic intrusives in the area. In most cases the roof of each seam is weak mudstones and shales. High levels of support are required during development and ahead of longwalls, and it is not possible to use place-changing in developments – by law and in practice it is essential to cut one cycle and then bolt immediately. Cycles are also limited to 0.9 m. The roof is normally bolted with six bolts of 2.4 to 2.6 m length, and each side is supported by two or three roofbolts of 1.8 m. Roof and sides are meshed. The width of roadways is limited to 5.2 m or less, due to difficult strata conditions and weak floor. Roads are driven at 2.6 to 2.8 m high in thinner seams and 4.0 to
4.5 m high in thicker seams. Major considerations are the height required to transport longwall shields and the area required to provide adequate ventilation.

All seams are gasy, with the gas content increasing from about 5-20 m³/t of total gas content, with depth. There is a theoretical risk of gas outburst at depth, but no occurrences to date. The seam gas is mainly methane. The greatest problem with gas is the extremely low gas permeability of Kuzbass seams. It is exceptionally difficult to predrain the coal in the target seam, even with closely spaced holes on suction. There are also numerous seams above and below the mining horizon which release large volumes of gas into the face and goaf areas. The deposits are world class, with good to moderate mining conditions that are well suited to long, wide longwall panels, and contain high quality coal.

**Longwall characteristics**

SUEK operates a total of twelve longwalls, including ten in Kuzbass. All of these produce high quality steam coal for export. Five longwalls operate in seams of 3.6 to 5.2 m and the rest are in seams of 1.9 to 2.6 m. The five thick seam longwalls operate in low ash coal and produce 5900-6200 kcal coal mainly without washing, but the thin seam mines all have access to washeries.

The only system for mining is longwall. There is no room and pillar mining in SUEK mines, and no real potential. Most Kuzbass mines operate a single longwall, but Kirova Mine operates two longwalls simultaneously as this is a thin seam mine extracting large reserves of high quality coal. One set works in a seam of 1.9 m and the other is 2.6 m. In 2017 this mine produced more than 7 Mt ROM, and output is expected to increase because one longwall set is being replaced at the end of the year, and the second set should be replaced with new equipment in 2019.

Longwalls are conventional, using two gateroads and leaving pillars between walls, so layouts are very similar to Australian longwalls. All main roads and longwall gateroads are supported with bolts, but cross-measure drifts must be supported by arches under Russian law.

Caving conditions are relatively favourable. Even where sandstones occur above the seams they are relatively weak, jointed and easily caved, so supports are typically in the range of 800-1100 t.

Most deposits are synclinal with steep gradients near the outcrops. Deeper parts of the deposits generally have dips of 3 to 10° but the shallowest longwalls in each seam have gradients across the face of up to 26°. All faces are driven generally along the strike, ideally with a small inclination along the gateroads so the faces retreat to the rise, causing water to flow back into the goaf rather than into the faceline and down the gate roads.

Face widths have been increased over the last 12 years, from initial widths of 180-240 m to 300 m and SUEK is currently increasing the width of all suitable longwalls to 400 m. The first 400 m face was extracted in 2017 and this established what is believed to be a world record during its third month of operation.

Panel lengths have also increased over time, partly as a consequence of deepening the mines, and partly due to investment in improved longwall equipment with increased working lives. Several of the Kuzbass mines are under major railway lines which cannot be undermined until a critical depth has been reached, so this has limited the extent of longwall panels. However, once the depth is adequate longwalls can be extracted below these features. A 400 m wide panel that is 5 km long, and 4.2 m thick is currently being developed. This panel will contain approximately 11 Mt ROM, most of which will be of saleable quality without washing.
EQUIPMENT AND INFRASTRUCTURE

Most of the mines have been re-engineered and re-equipped in recent years and they are now modern mines that are comparable to successful longwall mines in Australia and the US. This has included twelve new main fan installations, five new longwall sets, two fully modernised longwall sets, plus major developments in gas management and mine safety. At the same time AFCs have been upgraded from a mix of Russian and imported machines to the latest imported models (PF4 and PF6 from Cat in thicker seams, Joy in thinner seams), shearsers have been upgraded from Eickhoff SL500 to Joy 7LS6 and Eickhoff SL900, and development has been largely changed from light duty Russian roadheaders to Joy and Sandvik bolter miners. Belt conveyor systems have been upgraded and are commonly 3500-4500 tph, originally using imported VSD drives and controls, but recently with high quality Russian designed VSD drives and controls.

Several mines have been converted from vertical shaft winding by driving inclined drifts with belt conveyors. SUEK now has only one mine that winds coal in a shaft and even this has recently been upgraded to a fully automatic operation to maximise capacity.

TRANSPORT

Owing to the steep gradients and soft floor and frequently wet conditions in most mines the most common system for transport of men, materials and longwall equipment is roof-mounted monorail diesel locomotives. These are significantly less flexible and efficient than rubber tired diesels, but they can operate on gradients up to 26° or more, and they are not affected by floor conditions or water, and they are narrow so can operate alongside belt conveyors. They have improved longwall salvage operations compared to the former methods of dragging shields along the salvage chamber using low speed winches. After the face is bolted up a monorail is installed through the face, so diesels can drive along the face, pick up pans and supports and drive out the other end. However, they are a major constraint in transferring equipment to the next longwall.

A monorail diesel carrying a 28 t shield or a 90 t shearer can only move at 1-1.5 km/hr, and it stops frequently at corners, shunting points and changes of gradient. In this mode of operation availability is low due to frequent breakdowns of the locomotives. Upon reaching the installation chamber the diesel drives along the new faceline and lowers the shields at the required positions, and carries on through the face to exit. In many instances a loop is established to enable several diesels to work without interfering with each other. Beams are installed using roofbolts and heavy brackets. These bolts are in addition to the roof support system. It is rare for bolts to fail, even when carrying large shearers (7LS6, intact but minus the drums), because anchors are installed to high standards, in the knowledge that they will be carrying big loads. An additional limitation of this technology is damage to beams. When shields, pans, AFC drives and shearsers are being transported they are drawn up close beneath the locomotive. Any swaying of the load translates into torque acting on the monorail beam, so the profile becomes twisted and damaged. To counteract this, it is necessary to set a steel beam against the roof, and then attach the monorail beam to this. Monorail beams have also been reduced in length in order to reduce bending and buckling damage.

VENTILATION

The system of ventilation in SUEK mines is forcing, or push-pull in the larger mines. Forcing main fan ventilation is necessary as the ventilation must be heated at every point where air enters the mine. In winter the atmospheric temperature is -25 to -40°C, so air has to be heated or the intakes would be sealed with large accumulations of ice. At each fan site large boilers are installed, and by law these must be operational from September to May. They actually run at full power from October to April, and during this period they use approximately 10 MW of power to heat each 10,000 m³/minute of ventilation flow. Most mines require 11-25,000 m³/min of air to ventilate (~200-400 m³/s) so this means 10-20 MW of heating power for each mine, 24 hours a day for about five months a year.
In recent years SUEK has constructed 12 main mine fan installations, each complete with a boiler, or large electrical heating arrangements. In recent years the designs have changed from conventional large concrete fan houses to compact, containerised units to reduce construction and installation time. Each unit produces 5000 \( m^3/min \) and banks of these modular fans are installed on top of short vertical shafts.

Standards of ventilation are very high, partly because the mines are gassy, and partly to satisfy Russian mining laws. In addition, areas that are accessed by diesel locomotives require 1000 \( m^3/min \) of airflow (>15\( m^3/s \)) to dilute fumes. This is a major constraint as mines require four to twelve development units, all of which have to be supplied by monorail diesels – a total of 60-180 \( m^3/min \) of air just for developments. Longwalls also require high flows of 1500 to 3000 \( m^3/minute \) (25-50 \( m^3/s \)). Gas emissions for the Kirova and Yalevskovo Seam 52 longwalls are 170 \( m^3/min \) and 130 \( m^3/min \) equivalent pure methane (2.83 and 2.16 \( m^3/s \) respectively). Without methane drainage the longwalls would require 283 and 216 \( m^3/s \) of air to dilute the methane to 1%, which clearly is an impossibility.

**GAS MANAGEMENT**

All mines in Kuzbass are gassy, however emissions on LW5003 were relatively low due to the shallow depth (to 260 m) and some degassing due to over-working in Seam 52. However, the unworked Seam 51 is 26 m above, and there are seams within the zone of influence below, so gas emissions are substantial. Gas permeability is extremely low (0.01 to 0.001 millidarcy) in this area, so most gas is released, by mining-induced destressing and by caving, directly into the mining area and the goaf just behind the face. Typically ranges of in situ gas content in this mine are 5 to 11 \( m^3/t \). So even if gas emission during mining was only 3 \( m^3/t \), this would require more than 250 \( m^3/s \) of ventilation to dilute it to 1% or less, which is the Russian maximum. Clearly it is not possible to pass this amount of air through a longwall face, so extraction of gas from the goaf at high concentrations is essential. This is achieved by drilling vertical boreholes of 156 mm diameter ahead of the face. These are cased to approximately 10 m above the seam as they pass through the goaf of the overlying seam.

For LW5003 two rows of boreholes were used, one row about 30 m from the bottom gate and the second, wider spaced row 120 m from the bottom gate. The direction of airflow through this face was top to bottom, so the bottom gate (Conveyor Road) is the return airway. The lowest pressure is at the junction of the Conveyor Road, so this is the direction of gas migration. In some faces the return airway is the uppermost road, so the gas wells are located near that road. These are shown in Figure 2, along with the general layout of the mine and the design of the longwall panels. The thin lines across the panels at right angles show the monthly retreats, and the angled, double lines across the panels are narrow roads which are needed under Russian law to provide escapeways.

Two boreholes are kept on suction at all times. The life of a borehole is short so the vacuum pumps are kept on the holes closest to the face. There is also a 700 mm diameter borehole located near the face start line which draws high concentration gas from deep in the goaf. All these systems extract methane at 80-90% concentration. There is also a system to drawn gas out of the goaf from behind the face, using inclined holes drilled through the sealed crosscuts.

Gas monitoring is local plus centralised and is maintained in all intakes, all returns and in every development and longwall working area. On a longwall face detectors are placed at the intake end of the face, return end, blind end (the collapsing roadway at the return end of the face, but outside the face airstream), and two detectors on the shearer. There are also airflow detectors, airborne dust detectors and carbon monoxide and smoke detectors. Under Russian law these must automatically trip electrical power at 1% methane. Within SUEK the detectors trip power at 0.8% and sound alarms at 0.6%.

All gas monitoring data is transmitted to the surface gas control room which is manned 24 hours a day by trained personnel. Their role is to detect rising trends, and monitor disruption of ventilation. These operators are adjacent to the Mine Dispatcher who monitors mining activity and directs activities in the event of any emergency. This ensures that in any
emergency accurate information on ventilation and gas monitoring is immediately available. The mine dispatch office also gathers data on all main items of equipment, such as pumps, main fans, conveyors and longwall equipment, so this is a safety and production monitoring centre.

Data from all the SUEK underground mines is then transmitted to the Regional Office which is also manned 24 hours a day. This provides an over-view of all activities and the status of production equipment. It also shows every warning and alarm from the gas monitoring system.

A similar dispatch room is maintained in the Moscow headquarters so that in the event of any emergency the specialists in Moscow can be fully informed, with access to all data coming from a mine and able to assist and support staff in the region and at the mine. This is partly due to the vast distances in Russia. Kuzbass mines are 4.5 hours east of Moscow by commercial plane. The most remote mines are 9 hours away from Moscow. In addition to these constantly monitored systems every gas alarm triggers an automatic system of SMS messages that ensures all senior managers are aware of a problem, and can commence monitoring it if necessary.

The seam plan also shows a common mining practice in Russian mines. “Diagonal roads” are driven right across the longwall panel in order to meet the requirement for escape on foot within the 90-minute life of the standard oxygen generating self-rescuers. No longwall will be approved to commence extraction unless these are in place. The longwall simply mines back through them. The monthly retreats shown on this plan demonstrate that there is no loss of output during this crossing. These roads are driven narrow and about 2.6 m wide (or seam height if less), and heavily bolted. Weak areas are cable bolted and steel beams are bolted on the roof across the road. They are driven on an angle so that when the longwall crosses the road only three or four supports are in the roadway at any time. Occasionally longwalls also cross multiple roads, which are parallel to the face, but these have to be heavily supported with bolts, cables and cribs.

At the top of the seam plan is a short longwall numbered LW5001, that contains 2.64 Mt of reserves. The depth of cover to the top gate on this face is only 20-40 m and about 80 m at the bottom gate. It is not economic to use opencast for this small area, so it will be extracted by the longwall. SUEK has successfully worked several such panels in recent years, and salvaged the equipment with difficulty, but without incident.

RUSSIAN MINING LEGISLATION

Mining legislation is extensive, and the State plays a major role in approval and administration of most aspects of coal mining, from vetting of annual mining plans to approval of all equipment that goes into a mine, continual inspection of operations, and occasional enforced
stoppage of mines. Licensing of reserves is very stringent and mining companies are forced to drill at close to 150 m centres before any longwall panel will be approved for extraction. Part of the reason is to accurately quantify all reserves, to enforce high levels of recovery. If coal is lost without good cause and prior approval then mining companies are fined several times the normal royalty. However, mining companies are not forced to extract coal that is uneconomic, but the process of detailed checks and approvals is designed to ensure best usage of the nation's mineral wealth.

Labour legislation has remained largely unchanged since the replacement of the Soviet system. Despite replacement of limited mechanisation and severe underground working conditions with the most modern mining technology an underground miner is limited to approximately 30 hours at the face per week. It has taken many years to obtain the right to work 8 hour shifts, rather than 6 hours, but this has not increased the permissible total working hours per man per week or per month. Working time is further reduced by miners being awarded approximately 8 weeks holiday per year. The result is that in order to keep one man on a machine, one shift a day, seven days a week, 52 weeks a year, Russian mining companies must employ 2.383 men – each working limited time with consequential limits to earnings potential.

**SUEK’S STRATEGY**

SUEK has developed a large and successful coal mining company based on the following strategies:

- Main focus is steaming coal
- High volumes
- Low sulphur, high CV, washed and sized and quality assured products.
- Control costs whilst increasing volumes.
- Control the full product chain – from mining, processing, rail transport, ship loading and marketing – in order to add maximum value and retain it.
- Apply best international technology, and develop innovative engineering solutions.
- Constant pressure for improvement in every aspect of the business.

This strategy has enabled SUEK to compete with Australian mines in export markets for thermal coal, despite the harsh climate, the need to transport coal thousands of kilometres to port, and the limited capacity and high cost of ship loading.

The company is still growing and increasing output. Several new mines are being developed to continue the increased production of high quality, high value coal. In order to sustain growth and continue to develop the most favourable deposits, additional reserve areas are being procured, explored and approved for mining. SUEK currently has 5.4 billion tonnes of JORC Reserves, with a substantial pool of resources that can readily be upgraded to reserves as required.

The core business is steam coal for Russian power stations and for export. Thermal deposits are structurally simpler than coking coal deposits so production is more constant and demand is also less cyclical. These factors create opportunities to achieve economies of scale. High volumes are essential in order to influence transport and port costs.

Cost have been tightly controlled, even in periods of strong demand and high prices. Increase in labour costs is directly linked to better productivity. Material costs are controlled by manufacturing all roofbolts, mesh, plates, monorail beam, conveyor structure and similar items in-house in SUEK owned factories. Miners’ earnings are lower than Australia, but there have been no redundancies in SUEK operations, and the company has increased production from 90.9 Mt in 2007 to an estimated 106 Mt by the end of 2017.

SUEK is in the value-added business. All coal is processed and blended, to maximise sale value and minimise transport costs per unit of energy, and sold directly to end users in order to retain value. Most of SUEK’s mines are in Kuzbass, far from the ports, so are highly dependent on efficient and competitively costed railway services. At the time SUEK was founded the Russian rail system had serious capacity limitations due to years of inadequate
investment, so it was impossible to ship high tonnages, and there were seasonal shortages of wagons. SUEK has addressed this by:

- Building new loading points to match the increasing output of mines.
- Building large marshalling yards to make up trains ready for dispatch onto the Trans-Siberian rail network.
- Purchasing large numbers of railway wagons, including larger, better wagons. The company now has a fleet of more than 30,000 railcars.
- SUEK works closely with the national rail company to increase the capacity of bottleneck trains in the system.

These measures mean that rail capacity is not normally a constraint to production. However, some mines have access quotas on competitors’ rail systems, and this occasionally necessitates trucking coal substantial distances to another SUEK train loading point, or creating large stockpiles, especially during periods of exceptional performance of mines.

SUEK is a major buyer of mining equipment and infrastructure. In the last three years the company has bought three new longwalls and extended two thick seam sets to 400 m, upgraded most AFCs, upgraded complete conveyor networks in several mines, purchased six new shearsers and 12 new bolter miners. All of this is world-class, high capacity equipment from US and European suppliers. The only exception is belt conveyors – here SUEK has worked with Russian companies to develop systems that suit local conditions and operate with software that has local support. As new equipment is introduced, changes in work practices and organisation are introduced to maximise performance and continue the drive for more output with higher margins.

**PERFORMANCE OF YALEVSKOVO LW5003**

The recent record performances were achieved by the team working on Longwall 5003. This was the second longwall panel in this recently developed seam. Seam 50 is approximately 45 m below Seam 52, which has been worked over the last 15 years with longwalls extracting approximately 5 m of coal. This new seam is 3.6 m thick, but with no significant stone bands, so ROM ash is less than 10% and the coal is suitable for export markets without washing. Seam 50 was developed by new inclines driven in seam from the outcrop at 14°, reducing to 9° as the seam flattens out. The full dip of the seams in the initial panels is approximately 8-12°, with the Conveyor Road of the longwalls approximately 70 m lower that the Ventilation Road (TG). These gradients present no operational difficulties. Table 1 shows the monthly output and LW5003 retreat.

<table>
<thead>
<tr>
<th>Period</th>
<th>Output (kt ROM)</th>
<th>Retreat (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 14-30</td>
<td>428</td>
<td>203</td>
</tr>
<tr>
<td>May</td>
<td>1407</td>
<td>560</td>
</tr>
<tr>
<td>June</td>
<td>1001</td>
<td>424</td>
</tr>
<tr>
<td>July</td>
<td>1567</td>
<td>699</td>
</tr>
<tr>
<td>Aug</td>
<td>850</td>
<td>364</td>
</tr>
<tr>
<td>Sept 1-15</td>
<td>56</td>
<td>24</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>5310</strong></td>
<td><strong>2274</strong></td>
</tr>
</tbody>
</table>

The longwall was commissioned mid-April 2017. The longwall gate conveyor has to elevate the coal 100 m from the face start line to the junction with the main inclines, and was 2.0 km long. The longwalls are laid out in this manner deliberately, in order to ensure that all water drains back into the goaf. This mine can be extremely wet. On a previous face in Seam 52 water inflows of 400 m³/hr were experienced on the face and the AFC was running more than 1 m below water at times. The resultant wet coal had to be dried on the conveyors to prevent it sliding back once it reached the main inclined conveyors. This included a tripper with a grizzly. Coarse coal went across the grizzly and right back onto the belt, and the fine, fluid material was collected in a tank, taken into a crosscut where it was treated using hydrocyclones. Ultra-fines that passed through the hydro-cyclones were collected by settling and
mechanical elevators and the fine solids were fed back onto the belt, on top of the bed of coarse coal at the next crosscut. This was successful, but experience has shown that longwalls must retreat to the rise in this part of the deposit so that any periodic inflows of water that occur will not inundate the working area and run down the conveyor gateroad.

This is the second panel in Seam 50. The first panel was 300 m wide and operated with a PF6 AFC and an Eickhoff SL500 shearer. It achieved more than 1 Mt for two months in the middle of the run, but the under-powered shearer was a constraint. The second panel was developed for 400 m width, and the AFC was extended and additional power added. An Eickhoff SL900 was procured to provide additional cutting power to enable high cutting speeds to be maintained and to allow the face to operate bi-di, producing up to 3500 tph. The web on this face (and most longwalls in SUEK) is only 0.8 m, due to the limited stand-up time of the roof, so the production is 1575-1700 t per shear, depending on seam thickness. Output of 1 Mt/month requires an average of 21 shears per day, or an average of 57 minutes per shear, 20 hours per day, every day. The peak monthly retreat was 699 m – over 20 m every day. The most impressive aspect of this performance was the consistency of output. It is an average of 1.072 Mt/month for the 4.5 months of production, and 1.325 Mt/month for the three full months of highest output, and equivalent to an average of 1.13 Mt per month from commissioning to bolt-up.

It was also noteworthy that very high levels of safety were maintained throughout the full period. There were no reportable injuries, no infringement notices issued and no reduction in standards. The operation was closely monitored by continuous gas monitoring systems, which would trip the power at 0.8% methane, and frequently inspected by senior staff and the State inspectors. No cutting of corners was permitted.

SUEK’s miners are paid a substantial productivity bonus from an agreed scale, subject to meeting quality specifications and complying with safety regulations. In this case the productivity bonus payments were substantial, and the key personnel were recognised and given personal awards. This serves to motivate other teams to emulate the success.

CONCLUSIONS

SUEK is achieving world-class levels of production, productivity and safety in underground mines. Similar progress has been made in washeries, opencasts, transport operations and the company’s export ports. The company follows bold strategies, is not affected by short-term consideration of share prices, and has continued to invest in holistic improvements even during the worst years. There is still considerable improvement under way, with two new underground mines being developed at present, and several longwalls planned to be upgraded and extended to 400 m. There are still problems to be overcome, including the constraints imposed by monorail transport and a major challenge to substantially increase development rates in order to replace longwalls. However, these are being actively addressed, and solutions will be developed. The company is profitable. In 2016 the EBITDA was US$965 million, or 24% on total revenue. Output increased by 8% overall with all of the increase being from underground mines - which produce premium quality products. Productivity increased by 12% in the year to 498 t/man month, and the amount of coal that was washed increased by 4 Mt, or 12% as a result of upgrading existing washeries and commissioning of a new 3 Mt/yr plant.

Technical improvement of mines is essential but improved equipment and operations alone are not adequate. It is essential that this is combined with clear and sound strategies for operational efficiency and commercial optimisation. In Russia mining and processing is only half of the equation, and transport and ship loading are of equal importance and complexity. Only a large company that can develop holistic solutions can be truly successful on the world scale.

SUEK is positioned to remain one of the lowest cost, long term coal producers in the market. The future of coal mining is not rosy, but if seaborne markets do contract then it is likely that the lowest cost suppliers will outlast the others, so SUEK can afford to plan and invest for the long term.
ANALYSIS OF BREAKAGES OF LONGWALL POWERED SUPPORTS - WHY CYCLE TESTING DOES NOT GUARANTEE HAPPINESS

Peter McInally

ABSTRACT: In recent years, SUEK has suffered serious breakages of four sets of powered supports, including caving shields and the attachment of the powered supports to the AFC. All of these supports had undergone prototype testing prior to manufacture, but the design errors were not manifested under conventional test conditions. Breakages occurred underground where it is not possible to instrument the equipment, and where reliable data is hard to obtain by visual means. This paper describes the "forensic engineering" that led to the identification of the root cause of the damage, and the ways in which this was rectified. Two new sets of supports have also undergone design changes to prevent the same risk of damage. The solutions are simple, but developing this knowledge has cost a lot.

INTRODUCTION

Siberian Coal Energy Company (SUEK) currently operates eleven longwalls, with nine in Kuzbass in Siberia and two at Urgal in the far east of Russia, with a mix of equipment. In recent years older shearers and AFCs have been replaced with Eichhoff and Joy shearers and AFCs have been upgraded mainly with DBT PF4 in seams of 1.6 to 2.8 m, and PF6 in thick seam mines extracting 3.6 to 5 m). The coal cutting and conveying equipment is all modern, imported machinery, including one state of the art Eichhoff SL900 shearer working in a 3.6 m seam, and a second machine currently on order for a 5 m seam. SUEK owns 2,455 longwall supports in two categories – 1.6 to 2.6 m, and 3.5 to 5 m sets. The company operates six to seven thin seam longwalls and four or five thick seam. One mine extracts both thin seams and thick seams, hence the variation in numbers from year to year. Of these supports, 170 are awaiting refurbishment, and one new set is being installed, but all others are currently in use.

Since 2007, SUEK has been modernising and extending longwalls and approximately 1,500 individual supports have been purchased in the period 2007-2017. There is currently only one set of Russian made supports remaining in the fleet and this will soon be replaced with a set of imported, thin seam supports. All other supports are imported models from Joy, DBT, Tagor, the old Glinik and Famur/Glinik of Poland.

SUEK’s program of modernisation and optimisation is continuing and during the next five years it is likely that SUEK will purchase additional supports to extend another two faces from 300 to 400 m, and replace two old sets of thin seam supports and one set of thick seam supports.

SUEK has been a major purchaser of powered supports throughout the last ten years, with substantial numbers of supports purchased from several manufacturers, and this will continue as old sets are replaced with high capacity, modern sets, and as new mines are brought into production. The company will continue to increase production, especially from longwall mines that produce high quality coal for export.

During this process, SUEK has suffered several instances of major breakages, each involving large numbers of supports. In each case SUEK personnel investigated the problems and identified the root cause, then worked with the suppliers to rectify the faults. The knowledge gained from these investigations has recently enabled potentially serious design defects to be eliminated in two batches of supports during the design review process – a full face set of

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2 The content of this paper reflects the opinions of the author. These are not necessarily the view of Siberian Energy Coal Company (SUEK).

"Old" names of equipment suppliers are used throughout, to reflect the name of the OEM at the time of purchase and for ease of recognition.
supports for a highly productive mine and a set of 58 thick seam supports for extension of a longwall from 300 m to 400 m.

Design defects rarely occur in supports from traditional suppliers such as Joy and DBT, as these companies have accumulated vast knowledge over many years, partly by maintaining extensive contact with users, and by deploying mining engineers to monitor operational aspects and enhance designs. Companies that are considering purchase of supports from new suppliers from Poland and China cannot safely assume that the same philosophy of producing equipment suitable for mining, rather than producing mining equipment, applies to the same extent. Joy and DBT (and their predecessor companies) have operated in a fiercely competitive, customer-led market for 50 years. Polish and Chinese suppliers do not yet have the same proven track record as they have largely operated in a different environment, which has not provided the same level of market feed-back and robust criticism of any minor defect.

SUEK is still buying supports from JOY and DBT, but has also ordered five full sets and two partial sets from three different Polish suppliers during the last ten years. When purchasing such large numbers of supports it is not possible to ignore new suppliers and to maintain a closed shop approach. This process has resulted in some difficulties, but it has enabled SUEK to modernise a high proportion of the company’s longwalls, and to produce substantially more coal - despite some losses because of equipment breakage. Most importantly, SUEK has worked with the supplier of the recent face sets to find the root cause of each problem, rectify it, and then to eliminate similar problems from further sets at the design stage. The supplier has responded very positively both to rectification of damage and to implementation of design changes. On three occasions, large test rigs have been built to SUEK’s design to physically prove amended designs, and these have provided real insight and accelerated the development of designs.

A basic understanding of the cause of failures is all that is required to enable other users to check designs, identify potential for failure, and to force a supplier to modify designs if necessary. Understanding the risks will also enable purchasers to specify suitable computer modelling of designs and/or physical testing of prototypes within their contract in order to enforce the necessary checking by the supplier. Finally, if a breakage does occur, then this knowledge could speed up resolution and resumption of normal operation of a longwall.

**HISTORY OF BREAKAGES OF SHIELDS**

During the last ten years, a number of sets of powered supports have failed in operation in SUEK’s Kuzbass mines, including the following breakages:

1. 2008 November 7th mine: Polish 2.4-5.0 m supports with Russian AFC. Lugs ripped off approximately 100 AFC pans before the problem was rectified by fitting different clevises.
2. 2008 Krasnoyarskaya mine: A new set of 1.5-3.2 m supports from the same Polish supplier was delivered shortly after the set in November 7th mine failed. Based on analysis of failure of the earlier set these supports were modified before going underground, and successfully prevented breakage of the AFC.
3. 2009 Krasnoyarskaya mine: After six months underground, the same Polish supports were found to have extensive cracking on the upper part of the under-side of the caving shields. Repaired by over-plating the cracked areas with 20 mm of high strength steel under warranty during the first face transfer.
4. 2010 Krasnoyarskaya mine: The reinforced caving shields on the same set were found to have broken again. Extensive repairs were required to remove damaged areas and rebuild the caving shields during the face transfer.
5. 2016 Polisaevskaya mine: A modern set of 1.2 to 2.5 m supports was purchased from a Polish supplier for a fully automated longwall with a working height of 1.6 m. The same company supplied the supports and the AFC. Within four weeks and 40 metres of retreat, approximately 20 lugs had snapped off the cast AFC line pans. The failures continued and by the time the panel had mined 1 Mt, every line pan was broken. The only exception was inspection pans, which took longer to suffer damage. SUEK established that the failures were due to a design fault on the powered supports, rather
than any defect on the AFC, and proposed modifications which were made by the supplier under warranty. The supplier replaced all line pans and since then the equipment has mined 2.6 Mt without any further failure.

6. 2017 Kirova mine: SUEK ordered a 300 m set of high specification supports to extract a 2.6 m seam. Kirova is an important mine extracting 7 Mt/yr mine of semi-soft coking and premium thermal coal. This set of supports was sourced from the same Polish supplier as the 2015 Polisaevskaya supports. Analysis of the design showed that similar design defects to the Polisaevskaya set were present, and the AFC lugs and relay bars would be subjected to excessive stress and could break. Modifications were required by SUEK, and the prototype testing was expanded to include testing of the snaking of the pans, using a rig SUEK designed for testing of the 2015 modifications. The modifications were optimised using this test rig, the set was manufactured and the supports are being installed in December 2017.

7. 2017 Yalevskovo mine: SUEK ordered 58 thick seam supports to extend the longwall in Yalevskovo Seam 52 from 300 to 400 metres. These were also from the same supplier. Design flaws were detected before the contract was signed and SUEK insisted on extensive design change to the base of the supports. This was done and the supports were approved for manufacture and testing of the prototype.

During the same period, SUEK purchased 114 supports from DBT, 422 from Joy and 420 from the original Glinik Company, before it became part of Famur Group in 2012. No significant failures have occurred on any of these supports. Every set that failed had been tested on a rig for 30,000 or more cycles, in full accordance with the European Standard EN 1804. Prior to this, all designs had been checked by finite element or similar modelling by the supplier.

It is significant that most of the failures were of parts of the supports that are not tested by EN 1804, such as relay bars, base steering rams, and the AFC advance mechanism. The only failure associated with roof loading was the breakage of caving shields at Krasnoyarskaya mine. This was extensive and required all supports to be extensively repaired twice, so it was of considerable concern that it was not identified during cycle testing.

**Reaction to major failures of equipment**

When anything big, strong and expensive breaks an equal and opposite reaction generally occurs:

1. The supplier blames the user for doing illogical and impossible things to his fine equipment.
2. The user proves he is not responsible for abuse, but tells the supplier he will sue for the full value of lost output – regardless of the contract. Relations are fraught and time is lost.
3. The supplier blames the materials – despite the quality assurance (QA) system and certification, which he had previously boasted about. He wastes weeks looking at metallurgical analyses, microscopic inclusions and the like, but finds nothing significant.
4. The last possibility that is considered is that the breakage simply means that the applied stress exceeded the strength of the material - because the stress acting on the failed component was far higher than the designers anticipated.

Considerable time and output can be saved by first considering the possibility of excessive stress - as this is logically the most likely cause of failure. Longwall equipment is so robust that even serious abuse should not be able to break it - and abuse is hard to hide. Modern QA systems are extremely robust, so the chance of a whole set of caving shields or a complete set of AFC pans being made from the wrong material or with the wrong technology, is slim to non-existent. Welding processes are also formally defined and thoroughly checked, so weak welds are not common. In most cases weeks of lost or reduced output could have been saved if the most likely cause had been considered first, instead of last. In the case of the failed caving shields the FEA showed that the area where major cracks developed should have had stress levels of only 60 MPa, but the supplier still postulated that welding defects or
deficient materials was the cause of the failure. If the cause is found to be stress that exceeds the levels anticipated by the designers, then the source of this stress and the mechanism that concentrates it can be quickly be identified and action taken to minimise production loss.

Stress and stress multipliers

One reason for resistance to the concept that excessive stress could be the culprit when enormously strong components are broken is that the associated mechanism has limited power. For example, when AFC lugs broke engineers, assumed that this could not be due to excessive stress because the thrust of the advance ram was only 400 kN during conveyor push and 650 kN during support advance. To break a lug in tension probably requires a tensile force of the order of 5-8000 kN, and this clearly cannot be generated by the ram that drives the relay bar. Furthermore, during advance of the support, the loading on the lug is limited to the force required to move the support, so it is probably less than 200 kN in most cases, occasionally increasing if advance of a support is impeded. Even so, each support acts alone during advance and neighbouring supports cannot assist a jammed support to move.

However, AFC pans do not behave in isolation, so it is possible to generate high forces during AFC push-over. During a 15 pan snake, 15 rams are pushing. If one pan cannot advance and rotate to form the snake (because it has hit a step in the floor, for example), then the surrounding pans feel this reaction and all 15 rams act to act to assist the blocked pan to force it to advance. This means that the force acting on the lug of the blocked pan may be 15 x 400 kN, rather than the 400 kN that is directly connected to it, and as this force is applied a considerable distance away from the lug it generates torque instead of linear stress. Therefore the stress may be far higher than originally foreseen.

Furthermore, the direction of stress may be reversed. Take the hypothetical case of one pan that cannot move through the snake because it is obstructed, or because the advance ram is bypassing and cannot produce force. Immediately the thrust of all the other activated rams will act on the pans adjacent to the one that is hanging back and will drag the blocked pan forward. In this case the AFC lug on the blocked pan will experience very high levels of force, but its direction will be reversed and the stresses in this lug will be tensile rather than the normal compressive stress. There will also be massive bending forces or shear forces acting on the lug as the neighbouring pans force the blocked pan to rotate in order to form the curve in the snake because pans can move forward only if they rotate.

It is also possible to multiply other forces so even a small force can produce high stress. The two most common ways of doing this are levers and wedges. A wedge with a face angle of 1 degree will multiply a force by approximately 50 times, so shallow angled contacts between components can multiple force greatly and generate extremely high levels of stress. Levers also are force multipliers. When an AFC pan is dragged forward by three or four pans on either side, these are acting as levers and they generate very high torque, which acts on the point of blockage. Another consideration is the direction of the force. It is expected forces will be compressive or tensile because the advance rams act at 90° to the AFC and are linear mechanisms. When the conveyor is pushed it “should” create compressive forces in the AFC lugs, and when the support is pulled in everything “should” be in tension and self-aligning. However, it is possible that during conveyor push, extremely high forces can be generated which act at 90° to the axis of the relay bars. This can occur only if the lug on one AFC pan becomes blocked and cannot rotate to form the snake – because the relay bar is incapable of following the trajectory of advance and rotation of the AFC lug. If this does occur then extremely high levels of lateral stress generated by multiple rams and long levers can act at 90° to an AFC lug, and break it in bending or shear.

Cost of failures

All failures have a financial cost, even if it is only the cost of removing and replacing damaged parts. However, when longwalls are averaging in excess of 1000 t per operating hour, every
Stoppage costs thousands of tonnes and substantial revenue in the financial year. In some instances, the longwall has to stop immediately, while in others, the rate of production is reduced. In every case of breakage, the time required to transfer the longwall equipment to the next panel is protracted by the need to transport and repair equipment.

A conservative estimate of the loss of production time and tonnes of ROM arising from the failures of SUEK longwall equipment in the period 2007 to 2017 is shown in Table 1. Losses exceed 3 Mt of coal. The greatest losses were associated with the new thin seam set in Polisaevskaya mine in 2015 and 2016.

Table 1: Direct loss of ROM due to equipment breakages

<table>
<thead>
<tr>
<th>Mine</th>
<th>Cause</th>
<th>Year</th>
<th>Monthly output (Mt)</th>
<th>Lost time (Months)</th>
<th>Lost output (Mt ROM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 7th</td>
<td>Reduced tempo, plus repairs and fitting double articulated clevises. During face transfer cut off all lugs and replaced with double lugs in order to weld onto metal that was not fatigued.</td>
<td>2007</td>
<td>0.35</td>
<td>2</td>
<td>0.7</td>
</tr>
<tr>
<td>Krasnoyarskaya</td>
<td>Delayed start to modify attachment of AFC and relay bar to increase articulation at attachment pin</td>
<td>2007</td>
<td>0.28</td>
<td>0.7</td>
<td>0.2</td>
</tr>
<tr>
<td>Krasnoyarskaya</td>
<td>Removal of all supports from the mine to repair caving shields. Twice.</td>
<td>2009/10</td>
<td>0.28</td>
<td>3</td>
<td>0.84</td>
</tr>
<tr>
<td>Polisaevskaya</td>
<td>Lost output due to AFC downtime and loss of tempo</td>
<td>2015</td>
<td>0.3</td>
<td>3</td>
<td>0.9</td>
</tr>
<tr>
<td>Polisaevskaya</td>
<td>Transport AFC out and new one in to mine, plus repairs to relay bars</td>
<td>2016</td>
<td>0.3</td>
<td>1.5</td>
<td>0.45</td>
</tr>
<tr>
<td>Polisaevskaya</td>
<td>Repair of impact damage to several bases. Inspection of all advance mechanisms for damage.</td>
<td>2016</td>
<td>0.3</td>
<td>0.33</td>
<td>0.1</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>10.53</td>
<td>3.19</td>
<td></td>
</tr>
</tbody>
</table>

Average ROM loss in each affected Year (Mt) 0.66

**NATURE OF FAILURES**

**Types of failure**

There have been three main types of damage to longwall equipment in SUEK mines in the period 2008 17:

1. The heavy lugs that attach the relay bar of the powered supports to the AFC broke. The nature of the breakage has been slightly different, depending on the AFC design. There was also associated damage to the outer end of the telescopic relay bars because of excessive lateral force. The root cause was inadequate provision for lateral movement of the relay bar during AFC advance. Contributing factors were; excessive width of the relay bar, inadequate base steering as a result of incorrect positioning of the base steering rams, and errors relating to the design of the “steering wedges” on the relay bars and the front inner section of the bases.

2. Heavy brackets inside the base of supports that attach the AFC advance mechanism to the base bridge were ripped off on a small number of supports, and impact damage occurred on a substantial number of AFC advance cylinders. This was due to the design and positioning of the mechanism to hold down the relay bars.

3. A full set of caving shields developed multiple, large cracks in plates and welds as a result of interference between the top, outer edges of the upper lemniscate link with the vertical ribs of the caving shield. The two components were forced together at a very shallow angle as the supports yielded and converged. The root cause of the breakage was failure to include a 10-12 mm spacer on the hinge pin on both sides of the lemniscate link to maintain clearance between the sides of the caving shield and the sides of the lemniscate linkage during convergence of the supports.
Breakage of AFC Lugs by PRS

SUEK has experienced failures of AFC attachment lugs on two AFCs since 2007, with a total of more than 200 lugs broken. Typical results are shown in Figure 1, for a welded AFC, and Figure 2 for a cast AFC. In both cases the system included an articulated clevis, as shown in Figure 3. The vertical pin of the clevis allowed approximately 8° of articulation in both directions – which exceeds the maximum rotation of the AFC pan in the middle of the snake. The other point to note is the very limited clearance between the attachment and the AFC lug – presumably because the designers believed that the swivel on the clevis could accommodate all necessary articulation. In both 2008 and 2015, the AFC lugs started to break within days of commencement of operation and 50 or more pans were damaged in the first few months. In both cases inspection of the adjacent, unbroken lugs did not reveal any signs of defects, excessive loadings or bending or deformation. The lugs then broke suddenly and with no prior permanent bending or cracking. This indicated that the cause was a loading condition that only occurred intermittently and whenever it occurred it broke the lug. This opinion was reinforced when breakages were found with half the broken surface showing slight rust, and the other half with a clean, fresh break, indicating two discrete failure events, at least several hours apart and not more than one or two days apart.

![Figure 1: Breakage of welded AFC attachment lugs – 2008](image1)

![Figure 2: Breakage of cast AFC lugs 2015 (every line pan was broken in this manner)](image2)

![Figure 3: Clevis and attachment to AFC](image3)
The first major failure was extensive breakage of the lugs on a new, Russian-made AFC, which was attached to a new set of Polish roof supports. Because two manufacturers were involved, each blamed the other. However, a short period of observation underground proved that the damage occurred during push-over of the AFC and that the fault lay with the design of the powered supports. The clevis was redesigned to allow more articulation at the connection with the lugs, in order to prevent lock-up of the mechanism. The root cause was excessive width of the relay bars which meant the relay bar collided with the base as the AFC was pushed over. Unfortunately, the same problem occurred with a new longwall purchased from a different Polish supplier in 2015, and the same defective design was subsequently proposed for another full set of medium seam supports and for a 100 m extension of a thick seam set.

Two AFCs have been destroyed, and another was modified before going underground to prevent similar failure, plus the design of one full set and one partial set of supports had to be substantially modified at a late stage in design. Therefore, it is worth understanding this problem and the solution.

SUEK has had no breakages with structures on Joy or DBT supports operating in the same conditions, with the same operators during this period. Their designs have eliminated the root cause of these defects. SUEK also has one full face set and one partial set of 68 supports, from the original Glink Company to a well-proven design. These supports were supplied in 2010 and 2011, and no damage has occurred. The root cause of the design fault in two makes of Polish supports was failure to allow adequately for lateral movement of the AFC pans during snaking.

As a pan is pushed forward, the pan joint on the goaf side opens up and the pan rotates. This causes the attachment lug to move along the face and the relay bar must be able to follow this lug. If it hits the internal sides of the base tunnel, it can impede the rotation of the AFC pan, even if there is articulation available at the pin connecting the relay bar and the lug, or even if there is a swivelling clevis between the end of the relay bar and the lug attachment. This is shown in Figure 4.

![Figure 4: Lateral movement of AFC and relay bar](image-url)

Once the relay bar cannot move laterally, the system is almost locked. If there is clearance between the sides of the attachment and the AFC lug, then the lug can slide along the pin,
but once the lug comes into contact with the side plates of the relay bar attachment, the system is fully locked, and severe stresses will be generated if the AFC pan continues to rotate.

The cumulative lateral movement of the AFC lug/relay bar connection point is substantial. It varies depending on the length of pans, the width of pans (including depth of the lugs, to the attachment position) and the detailed design of the face side pivot points. Modern AFCs have a stepped joint at the face side, and this may mean that the trajectory of the pan is different; depending on the direction the snake is formed.

In the convex part of the curve the relay bars move in one direction, and in the concave section of the snake they move back again, until they return to their original position, in line with the centre of the support.

The first half of the snake, for a SUEK longwall with a set of Polish supports and AFC from the same company is shown in Error! Reference source not found.. The push-over on this face is 1 m, although all other SUEK faces operate with 0.8 m webs. The design of this support had to be modified as this drawing and subsequent testing of the prototype

![Figure 5: 15 pan snake of 1 m web - showing lateral movement of the AFC attachment points in the convex part of the snake](image)

This clearly showed that the AFC would be broken - unless the relay bars or clevises broke first. The AFC attachments needed to move a total of 97 mm along the face but the front end of the relay bars could not follow this. Note that the wedges are not in contact with the internal sides of the support base on support No.1 in this drawing, but they are in contact on supports 4, 5, 6 and 7. If they were shown in contact before the start of the push-over, then the relay bar would be trapped, and high forces would be developed.

The lateral movement during snaking of the AFC is considerable. In this case the lateral movement of the front end of the relay bar is 95 mm. The lateral movement decreases along the length of the relay bar from 95 mm to zero at the pivot point. The design must ensure that the relay bar can follow this amount of lateral movement of the AFC pan without locking up at the attachment point, and without any horizontal contact between the relay bar and the base of the support, other than at the attachment and at the designed pivot point, inside the base of the support.

The only way to guarantee this is to have considerable space between the sides of the relay bar and the inside of the base, and to install effective contact makers and breakers to ensure
that space is created when the support is pulled in, so that it is always available during AFC push.

There are two ways of achieving this. The best way is to use blocks inside the narrow portion of the base tunnel, close to the front bridge, with similar blocks on the relay bar. When the support is fully advanced, the tapers on the blocks of the relay bar force the base of the support to move horizontally. This creates a gap of the order of 50-60 mm between the main part of the relay bar and the base, when the contact makers are engaged.

As soon as the AFC push commences and the relay bar has moved forward approximately 50 to 75 mm, contact is broken. If “banana” slots are used on the AFC attachment lugs then contact is broken during the lost-motion phase, before any force is applied to the AFC and before advance and rotation of the pans commences. The second way is to use wedges, but these create less clearance and must be positioned well forward so contact with the base is broken at an early stage in the AFC push. This means that the internal faces of the base must be carefully designed to prevent contact. Deep wedges are required to ensure there is adequate room for the relay bar to follow the AFC, but they must be short in order to break contact before the AFC starts to rotate. It appears that contact with blocks inside the narrow section of the base is a considerably more effective mechanism, as it provides maximum free rotation of the relay bar. When combined with suitable internal angles of the base, it can guarantee that contact does not occur throughout the full push-over.

Until recently, Polish suppliers have used articulated clevises at the front of the relay bar. They appeared to believe this allowed the relay bar to follow the AFC, and provided drawings that showed the range of rotation of AFC pans throughout the snake, but without showing lateral movement – which cannot be accommodated by a clevis. When the Polisaevskaya AFC was broken, the designers’ proposed replacing the clevises with even more articulation. Unfortunately, once the relay bar comes into contact with the base of the support the relay bar cannot move sideways to follow the AFC pan, so even if the clevis still has 10° of articulation remaining, it cannot be utilised, and elevated stresses will be produced.

During this process SUEK proposed a design for a test rig that could physically prove whether collision will occur at any point in the conveyor push-over. This ensures that an AFC pan follows precisely the same trajectory as it would on the face when it is connected to 15 or more pans, and a full snake was formed. This is shown in Figure 6.

Figure 6: Conceptual design of rig to test pan snake
A modified AFC pan is placed onto a sheet of steel with curved slots that exactly replicate the trajectory of the centre of the pan and of the AFC attachment lug. The modified pan is connected to a powered support and pushed over the base sheet a distance of 500 mm, to replicate the first half of the 1000 mm push-over. In the second 500 mm of advance the AFC lugs return towards their start position, so there is no point in modelling this phase, as it cannot create contact. It is essential that the guide slots accurately represent the movement of the pans, using information obtained from the supplier of the AFC, which will be different for pans of different widths, lengths and different face side pivot designs. The rig used in the factory tests is shown in Figure 7. This also allows the base of the support to be rotated relative to the AFC to allow for the lead of the bottom gate of face, and for misalignment of supports. This rig enabled clearances and lack of contact to be checked throughout the full 500 mm of advance in the convex section of the snake. In the tests of the modifications for the Polisaevskaya supports, a swivelling clevis was retained. However, when a similar rig was used to test another, later set of supports in 2017 no swivelling clevis was used and additional clearance had been provided between the AFC lug and the vertical sides of the attachment to the relay bar, similar to the approach that has been proven by Joy and DBT.

Figure 7: Photographs of test rig replicating the snake of AFC in the supplier’s premises

Use of this rig revealed that the wedges on the relay bar had to be positioned close to the tips of the support base, so they could break contact as soon as the push-over commenced, and also demonstrated the inherent advantage of contact being made inside the base tunnel, instead of on the angled tips.

Base steering

In the case of the Polisaevskaya supports, the base steering rams also appear to have been a factor contributing to breakage of the equipment. This face was inclined, although only at 7-13°. The base steering rams did not appear to adequately correct the alignment of the supports relative to the AFC. The face operated with a lead of approximately 25 m at the bottom gate (Conveyor Road in Russian parlance), or approximately 5° lead, to prevent the AFC creeping down the face. This lead was effective as the head drive was maintained well clear of the bottom side of the Conveyor Road, so there was no creep. However, the orientation of the supports did not appear to be fully corrected during pull in. If the rear of the support is rotated downwards and not fully corrected, then the angle of the internal edges of the support will be less than the designer intended. This means that the angle between the relay bar and the inside of the base is decreased, and this increases the chances of the relay bar contacting the base during AFC push-over.
As SUEK successfully operates many inclined faces, and has operated on gradients of up to 26° it appeared that there was some difference in the steering arrangements on these supports. The Polisaevskaya supports have the base steering ram immediately behind the bridge and in front of the legs, and almost in line with the pivot point on the relay bar, which is 300 mm behind the bridge linking the two base pontoons. The steering of the supports is sluggish. Another SUEK mine has a face comprising 111 Tagor 15/22 supports along with 68 supports that were supplied by the original Glinik Company. The Tagor supports have the base steering rams behind the legs and the Glinik supports have them in front of the legs.

Inspection and discussion on the face revealed that the steering of the two types of supports was noticeably different. The Tagor supports started to correct as soon as the pull-in commenced, but the steering of the Glinik supports was slow to start, followed by a big swing at the end. The Glinik supports have the base steering ram in front of the legs, in the same position as the supports at Polisaevskaya mine. Both of these sets are hard to steer. All Joy, DBT and Tagor supports owned by SUEK have the base steering rams mounted behind the legs, and they steer progressively and predictably.

SUEK has recently purchased a new set of supports for Kirova mine and ordered about 60 supports to extend a face in Yelevskovo mine (Seam 52) from the same supplier as the Polisaevskaya set. Both designs have the base steering rams behind the legs, so it appears that the supplier has recognised that the location of the base steering rams does influence the ease of steering of supports.

**Impact damage**

The Polisaevskaya supports also suffered damage from internal collision between the AFC advance mechanism and the inside of the base. Damage included ripping out the bracket that attaches the AFC advance ram to the base bridge, severe deformation of the piston head on some advance rams, and deep indentations on the collars of the cylinders. The location and nature of the holding down lugs is shown in Figure 8 and the damage is shown in Figure 9.

![Figure 8: Internal lugs for holding down the relay bar](image1)

![Figure 9: Impact damage to AFC advance rams](image2)

The supplier responded quickly and made a full set of heavier mountings and imported them to Russia. Unfortunately, the approach was a repeat of previous experiences – breakages must be due to inadequate strength – redesign it and make it stronger. There were two downsides to this proposal:

1. The new mountings were certainly heavier and stronger but installation required the old mountings to be cut out, and the new ones to be welded on. This could not be done underground. Transporting the whole set to the surface for this modification would cost SUEK four to six weeks of production due to the severe limitations of underground monorail diesel transport of heavy equipment and the need to transport the shields to off-
site workshops for modifications. On site repair was not an option as the temperature was minus 15-25°C.

2. The designers had not identified the cause of the breakage, so they were designing blind. They had not carried out a survey to find facts, so even if the requirement really was strength, it is impossible to design and achieve an acceptable factor of safety unless the stresses are known.

SUEK carried out a survey of a number of broken supports that had been brought to the surface, and this established that the breakage of the mountings was due to the collar of the cylinder of the AFC advance ram impacting with the lugs inside the base that hold the relay bar down.

The advance ram is mounted with the piston at the front end of the support base, and the cylinder is attached to the rear end of the relay bar. The holding down lugs is located about 0.3 m behind the bridge. The evidence established that, as the ram closed during AFC push-over and the cylinder moved towards the front of the support, it occasionally collided with these lugs. If this happens then the ram will be blocked. This will mean that the neighbouring supports will try to pull the AFC pan (and the cylinder) forward. A total force of 4000 kN from ten advance rams acting to overcome this blockage could account for the observed damage - ripping metal off the cylinder, ripping metal off the holding lugs and serious deformation of the forged head of a piston.

Item 1 of Figure 9 shows obvious damage to the collar of the outer cylinder. This has been hitting the holding-down lug, shown as item 3, and gouging metal off it. Item 2 is the forged head of the piston on the advance ram and this serious deformation has occurred after only 2.5 Mt. The proposed ram mounting is shown in item 4. This requires extensive welding that could only be done on the surface.

The root cause was the design of the mechanism for holding the relay bar down. The broken supports were face end and transition supports, which had larger diameter advance rams, which greatly increased the likelihood and severity of contact. Inspection of the line supports revealed that approximately 50% had visible damage to the collar of the cylinders of the advance ram and to the lugs, but this was minor – with about 10-15 mm of metal ripped off.

SUEK had developed a mechanism to hold the piston head of the advance ram in place even if the original mounting failed, so it was decided to repair the face end and transition supports by fitting new mounting brackets, trimming the holding lugs down to increase clearance and replacing the seriously damaged advance rams under warranty, but to do a face to face transfer of the line supports. It was hoped that the bits knocked off the advance rams and the lugs would have created adequate clearance to prevent further damage. This avoided the significant loss of output associated with transport of all supports to the surface, and it has been successful. No further failures have occurred.

**Supports modified prior to manufacture**

The new supports for Kirova mine were found to have a similar arrangement for holding down the relay bar, consisting of lugs trapping the upper edges of the relay bars, located about 0.3 m behind the base bridge.

These are intended to prevent the rear end of the relay bar lifting when the base-lift mechanism is activated. This lifts the front of the support by pushing against the relay bar, but this steepens the angle between the advance ram and the relay bar. This increases the vertical component of the resultant force, and this acts at the rear attachment of the advance ram to the relay bar, lifting it vertically as it moves backwards relative to the base of the support, as shown in Figure 10. This vertical force must be counteracted by some form of retainer inside the base of the support, or lift at the front of the base will be reduced, and the rear of the relay bar could be trapped inside the support.
Figure 10: Resultant forces during support advance with base lift

The Kirova supports could not be changed to a rear hold-down arrangement as used by Joy and DBT because the bases are too short to allow an internal bearer plate to be used. A complete redesign, FEA and cycle testing of a new base was not feasible in the available time. Based on the limited damage to line supports on the Polisaevskaya set due to interference between the advance rams and the holding down lugs, it was decided to accept the design, along with warranty for damage. The hold down arrangement has been checked and clearances have been maximised as far as possible.

Another partial set from the same supplier for use in 4.5 m extraction was also checked by SUEK. In this case there was adequate height and length available to change these supports to rear hold-downs for the relay bars. The design of the base was changed completely as a result of SUEK’s concerns, with an effective contact make/break mechanism inside the base tunnel, and a similar system to DBT for holding down the rear end of the relay bar, using a “doghouse” with an internal bearer plate that is in contact with a boss at the rear of the relay bar and ram, as shown in Figure 11.
The function of the holding-down mechanism is to counteract the combined effect of the base lift ram and the retraction of the advance ram, to pull in the support. These forces form a couple that will lift the rear of the relay bar, thus reducing the area of contact between the relay bar and the floor, which serves to lift the base of the shield as it advances. If this is located at the rear of the relay bar then the base rotates about this contact point and the contact between the relay bar and the floor is enhanced by the base lift ram, raising the whole base and allowing dirt to pass underneath. The system using lugs a short distance from the base lift ram does not appear to be as effective as the doghouse and bearing block used by DBT and unless the advance ram is mounted with the cylinder at the front so that it is stationary, there is also a risk of collision between the lugs and the advance ram.

Joy uses hold-down lugs on one of SUEK’s sets, but these are located far back on the relay bar to maximise lifting of the base. The top of the relay bar is wider in the contact area for the lugs to trap it without any possibility of collision. They also reverse the advance ram, so the cylinder is static and the (smaller diameter) piston moves. This eliminates the possibility of collision and probably also facilitates the flow of dirt out the rear of the support.

**Broken caving shields 2007 and 2009**

Four months after a face set of Tagor supports was commissioned cracking was observed on the lower surface of the caving shields on four or five supports. A full, detailed survey was carried out and this showed that the majority of the caving shields had long, wide cracks at the corners of the access cut-outs. Initially most were in the corners of the access pockets, but with time these became more extensive and then cracks developed along the outer edges of the underplates. The steel underplates in the area where cracks occurred were supposed to be low stress areas, with only 6 MPa in plate that should withstand 500 MPa. The supplier’s FEA is shown in Figure 12.

![Figure 12: FEA plot of caving shield, showing failures in low stressed plates and welds](image)

Clearly the cracks were occurring at areas of excessive stress concentrations, but it was necessary to establish the source of the stress and to determine how it was being concentrated. The supports were in use at this time so each hypothesis had to be checked out underground. The roof in this seam was irregular and difficult to hold and the telescopic canopies were ineffectual, so the face was difficult, with supports angled, and with areas along the face where the supports converged by 1 m or more. There was also a tendency for the canopies to be set at an angle with the tips up and the rear of the canopy lower, which flattened the inclination of the caving shields.

Observation of the shields eventually showed that there was high-pressure contact between the top edge of the upper lemniscate and the inside edge of the caving shield. This was normally seen as deep scoring of the metal inside the caving shield, but on several occasions fresh interference was seen and photographed. This consisted of deep scoring of the metal,
and large areas of “bluing” due to very high temperatures. This damage rusted over after a day or so, so it clearly had occurred shortly before the survey.

A comparison of this area of the Tagor supports with supports at other SUEK mines quickly revealed that Joy and DBT supports had 10-12 mm spacer discs between the lemniscate link and the internal sides of the caving shield at the hinge pin. This clearly was designed to keep the sides of the lemniscate and the caving shield apart to prevent collision and creation of a stress multiplier. On some other supports the lemniscates were shaped like dog bones – wide at the hinge points, and narrower in the midsection. Both designs effectively prevent contact when the support is converging.

The supports on the face at Krasnoyarskaya Mine did not have either of these features, and were a close fit inside the recesses of the caving shield. These supports yielded and converged at a load of 900 t. If the upper lemniscate linkage and the internal or external side of the recess in the caving shield came into contact at a shallow angle during convergence then this would act like a very shallow angled wedge and modify the vertical forces so as to generate extremely high lateral forces which exceeded the strength of the fabrications. This would induce tensile forces in the underplates of the caving shields, cracking plates and breaking welds, as observed on most of the caving shields. This was clear evidence of the source of the stresses that damaged the supports.

The only solution was to cut off all underplates and replace them with thicker, higher grade steel, with redesigned cut-outs to reduce stress concentration, and to change operating procedures to maintain a high angle on the caving shields in order to limit contact. The supports have worked continuously without further breakages.

The lack of clearance between the links and the caving shield is clearly visible in the photograph at the bottom of Figure 13. This small design error resulted in breakage of every caving shield on line, transition and roadhead supports. A 25 mm narrower link with a 12 mm spacer on each side would have prevented this damage.

Figure 13: Examples of breakage of caving shields, 2008
EN 1804 TEST

SUEK has suffered from a series of breakages, and the associated loss of output. Every set of supports had undergone cycle testing, and EN 1804 was followed explicitly in every case. The problem is the limitation of the test procedure:

- The test is static
- It tests only the components that are loaded by the roof.

It does make the support converge when it yields and it does not test the other important functions of a powered support – push-over of the conveyor, pulling in the support, steering the support during advance, and interaction of the support and the AFC pans.

Static test

The cycle testing is static, in that the top of the test rig is set to an agreed height (the normal working height) and the base of the rig is fixed. The support sits in this space and undergoes loading, yielding, unloading, and reloading thousands of times. Blocks of steel are installed under the base and on top of the canopy to induce stress concentrations, bending and twisting.

This is significantly different from the process of cycling of supports underground in a mine. In practice, when a support yields, it lowers off. If a support is overloaded then the support may converge until the legs are completely closed, but more commonly a support yields several times before it is reset, and this yielding may lower the support 50 mm or 500 mm. This is a critically important difference.

There is no suggestion that EN 1804 should be scrapped and replaced by testing in a rig that loads externally and compresses the support throughout its full range. It is not necessary, and it would not work. However, engineers involved in procurement of longwall equipment need to understand that EN 1804 is merely a test of structural integrity under set conditions of loading, and nothing more. It tests the structural design and the quality of manufacture, but not the functions of the support underground.

The problems that can occur in real life are due to the fact that as a support is converged certain components may come into contact. If the support continues to converge, then massive stress concentrations will occur. The designer obviously has not foreseen these so they are not detected by FEA, and they cannot be detected by a static rig test.

They might be seen in a compression rig, but only if the test engineers and the end users understood the potential for collision and interference to occur and created the necessary conditions of convergence combined with misalignment. However, if the potential for interference and unplanned contact is recognised then it makes more sense to engineer it out, before one piece of steel is cut, rather than test for interference using an expensive compression test stand and an expensive prototype support.

In the case of SUEK’s Tagor supports the design could have been changed relatively easily – by narrowing the whole upper link, or by stepping it in by 20 mm on each side, immediately past the ends that fit on the pivot pins. Either of these very minor modifications could have prevented damage to the caving shields.

Tests only the components that are loaded by the roof

SUEK has learnt the hard way that it is essential to test more than the structural integrity of the canopy, base and caving shield and the longevity of the legs and stabiliser ram.

It is inconceivable that major parts of the structure will break into pieces. If a support has undergone computer stress analysis then failure during testing is normally limited to small or moderate cracks in areas where stress raisers have been overlooked in modelling, or inadvertently introduced during manufacturing. Modifications as a result of cyclic testing of the
prototype most commonly consist of changing some tight radii, altering a welding procedure or introducing additional stiffeners and the like. If a 100 mm thick plate of 550 or 600 grade steel snapped during cycle testing there would be a contractual crisis.

But in underground conditions SUEK has suffered precisely this - sudden and complete breakage of large numbers of 100 mm plates and castings associated with powered supports and serious damage to 170 relay bars on one face alone. In total, SUEK has had more than 300 failures of AFC connector lugs – a component that is massive and which is designed to have very high factors of safety. The cause is not incorrect operation, but incorrect design, which cannot be identified during normal testing.

Additional testing clearly is required to prove the secondary functions of powered supports including:

- Base steering, to ensure that the alignment of the support can be corrected during support advance and that correction is executed throughout the full advance.
- Serious misalignment of supports can result in contact during AFC advance.
- Checks on the travel of the advance ram within the base tunnel and analysis of the mechanism that occurs during base lift, to ensure there is no risk of collision – for line, transition and face end supports as the cylinder diameter is likely to be different).
- Check axial and horizontal travel of the relay bar during push over of the full web, using the actual trajectory of the AFC pan at the position of the banana slot or pin hole. Check that the attachment pin does not lock up, that any articulation point (if a clevis is fitted) does not lock up, and check that the relay bar has adequate clearance inside the base throughout the full pushover. High quality computer modelling can do this, but physical testing with a test rig has proved highly beneficial in the case of non-traditional suppliers, and has greatly accelerated refinement of the base and ancillary components of the supports.
- Check that the contact-makers act as effective contact breakers and release the relay bar from the base of the support during the “lost motion” phase of the AFC push-over.

These tests and inspections are in addition to the normal inspections of hoses, ease of change-out of advance rams and base pushers.

Some of these can be checked by proper 3D computer modelling, by proven companies such as DBT and Joy (Cat and Komatsu). However, the precise trajectory of a pan during snaking depends on the design of the pan, the length of the pan and the effective width of the pan – which is the distance from the pivot point on the face side to the point of attachment to the relay bar on the goaf side. The support designer is responsible for this modelling, but the AFC supplier should be contractually obliged to provide precise details of the pan trajectory, at the centre of the pan, and at the lug. This will enable accurate modelling, and it can also be used to make a physical test rig.

The effectiveness of the base steering can be readily checked, especially if more than one prototype is made. SUEK has required construction of at least one face end, transition and line support for final construction check and detailing of hose routes, light fittings, guards, controls, before full scale manufacturing commences. This makes it easy to check the correction of the base during support advance, and to ensure that the contact makers/breakers are correctly positioned.

The area swept by the advance ram during retraction (AFC push) and extension (support advance) can be checked visually at this stage to ensure that the ram assembly and the base cannot come into contact, at any possible position and orientation of the relay bar.

The ability to shed mud and rock out of the back of the support should also be checked at this time. This is especially the case when the ram is mounted on the bridge at the front of the base and the cylinder moves in and out of the base, as the collar of the cylinder may compress material against the support during conveyor push.

The combination of EN 1804 with effectual design checks and 3D modelling can eliminate defects in design, but only if the customer’s engineers truly understand the mode of operation.
and functional requirements of all ancillary items. Failure of these items can stop a longwall as surely as breakage of canopies or failure of hydraulic legs and rams. SUEK has learnt not to assume that every supplier has the complete understanding of what can happen to his equipment in the underground environment.

CONCLUSIONS

It is essential to fully consider the functional requirements, design and action of the base and every ancillary device on powered supports, such as relay bars, clevises, base lifting rams and relay bar trapping, and the mechanics of base steering devices, powered side shields, AFC creep arrestors, AFC vertical steering rams, banana slots, face sprags and face end base retention devices, if any of these are fitted. It is best to develop this understanding even when purchasing from a well-proven supplier, but it is essential to fully understand all primary and ancillary functions of powered supports if you are buying from new suppliers, such as Polish or Chinese companies, in order to avoid costly failures.

Suppliers should rectify any failures under warranty, but the mining company cannot be compensated for consequential losses. It is better to question the “obvious” than to accept designs and then find they are not fit for purpose. Remember caveat emptor – “Let the buyer beware”.

THE ABUTMENT ANGLE MODEL AND ITS APPROPRIATE USE FOR LONGWALL TAILGATE DESIGN

Mark Colwell*

ABSTRACT: In Australia and the United States the longwall gateroad design methodologies ALTS (Analysis of Longwall Tailgate Serviceability) and ALPS (Analysis of Longwall Pillar Stability) were developed to assist in the design of chain pillar systems to satisfy gateroad serviceability requirements. Important to the application of both methods is a reasonably accurate determination of the vertical load carried by the chain pillars at the various stages of the longwall extraction cycle. To calculate the load, use is made of Tributary Area concepts and the Abutment Angle Model. While Tributary Area Theory and the Abutment Angle Model generally assist in providing a reasonable approximation to a complex issue; chain pillar monitoring studies conducted in Australia over approximately the last 30 years strongly indicate that without the application of prudent engineering judgment significant errors in the calculation of the abutment loads carried by the chain pillar system can be made. This paper explores the appropriate use of the Abutment Angle Model for longwall tailgate design and details a number of case studies to highlight the issue.

INTRODUCTION

Where total extraction methods such as longwall mining are employed, failure of the rock mass above the extraction horizon occurs and a goaf is formed. The goaf is of reduced stiffness compared to the original rock mass resulting in a portion of the overburden load above the extracted panel being distributed to adjacent solid coal (i.e. unmined area or in pillars) away from the goaf edge. The distributed load is referred to as the abutment load. With respect to longwall mining it is the side abutment load (A, MN/m) which is typically calculated and then utilised in conjunction with Tributary Area concepts to determine the vertical load carried by the chain pillars at the various stages of the longwall extraction cycle.

Various methods have been used to estimate both the magnitude and distribution of the side abutment load about the longwall panel, including stress balance approaches (e.g. Wilson, 1973 and 1981) as well as the use of surface subsidence profiles (e.g. King and Whittaker, 1971, Choi and McCain, 1980 and Mills, 2001) to assist in the evaluation. In developing the ALTS (Analysis of Longwall Tailgate Serviceability - Colwell, 1998) and ALPS (Analysis of Longwall Pillar Stability - Mark, 1990) design methodologies, the abutment angle and load distribution functions were assessed based on industry wide underground investigations (e.g. vertical stress change monitoring of chain pillars).

However, irrespective of which method is used to calculate the side abutment load; chain pillar monitoring studies conducted in Australia over approximately the last 30 years strongly indicate that without the application of prudent engineering judgment significant errors in the calculation of the abutment loads carried by the chain pillar system can be made (particularly at the tailgate corner of the longwall face). This paper explores the appropriate use of the Abutment Angle Model for longwall tailgate design and details a number of case studies to highlight the issue.

THE CHAIN PILLAR LOADING CYCLE

The chain pillars that will experience the greatest vertical loading at any particular stage of the extraction process will be situated between two longwall panels. This obviously accounts for the bulk of chain pillars in a longwall extraction area (i.e. series of longwall panels).

To assist with subsequent discussion and the terminology used, reference is made to Figure 1, which is a plan schematic of a typical Australian longwall mining layout utilising a two-
heading gateroad system. Figure 1 depicts a fully extracted longwall panel, one currently being extracted and a third where the gateroads (Maingate 3 or MG 3 – ‘A’ and ‘B’ Headings) are still to be completed to fully delineate the longwall panel and chain pillars. ‘A’ Heading is generally referred to as the travel road along which men, materials and machinery will travel, while ‘B’ Heading is called the belt road where the conveyor belt is installed to transport coal from the longwall extraction face.

![Figure 1: Typical Australian longwall layout (using a 2-heading system)](image)

In a series of longwall panels, ‘A’ Heading typically serves two roles, firstly as the travel road of the current longwall panel and secondly as the tailgate of the next. For example, the travel road of Longwall Panel 2 (LW 2, refer Figure 1) will become the tailgate of LW 3. Therefore this travel road/tailgate is subjected to a series of changing geotechnical environments, moving from development (Position a) to the passage of the 1st adjacent longwall face (Positions b and c respectively) and finally being subject to the approach of the 2nd adjacent longwall face up to the tailgate intersection (Position d) with the travelling longwall face.

With reference to Figure 1 it can be seen that the chain pillars are also subject to a series of changing loading environments with the following terminology being used to describe each stage of the chain pillar loading cycle:

- Position a – Development Loading
- Position b – Maingate Belt Road (MGB or Front Abutment) Loading
- Position c – Maingate (MG) Loading
- Position d – Tailgate (TG) Loading
- Position e – Double Goaf (DG) Loading

It is during tailgate loading that the chain pillar (or cross-section thereof adjacent to the tailgate intersection) will experience the greatest vertical loading throughout its active life. Where the active life is the period of time during which the chain pillar is playing its role in helping to maintain satisfactory roadway conditions. ALTS and ALPS focus on tailgate performance (at the T-junction, refer Position d - Figure 1) as the design condition. Within ALTS the chain pillar index in relation to the TG Loading Condition is designated as the Tailgate Stability Factor or TG SF and with respect to ALPS it is referred to as the ALPS Stability Factor or ALPS SF.

Both the TG SF and ALPS SF are calculated in a similar manner to a pillar’s Factor of Safety (i.e. Pillar Strength divided by Average Pillar Stress). In developing ALTS, Colwell (1998) suggests the chain pillar TG Load is generally equal to Tributary Area Load (T) plus 1.5 A,
while the Pillar Strength (where \( w/h \) is the chain pillar minimum width to height ratio) is calculated using the Bieniawski (1984) equation where:

\[
\text{Pillar strength} = 6.2 (0.64 + 0.36 \frac{w}{h}) \text{ MPa}
\]

In reality there can be significant variations to the general TG Load = \( T + 1.5 A \) (MN/m) between coalfields, collieries and along an individual gateroad due to several factors; in particular variations in the panel/pillar geometry and natural strata variations. Also in utilising the TG SF it is vitally important to note that the TG SF is not a Factor of Safety. To undertake the empirical analyses associated with the ALTS research, numerical values (or indices) needed to be assigned to the various parameters that affect gateroad performance. The TG SF Rating allows for an assessment of the impact (or contribution) that chain pillar size has on tailgate performance.

**ABUTMENT ANGLE MODEL**

To calculate the load carried by a chain pillar at the various stages of its loading cycle use is made of Tributary Area concepts and the Abutment Angle Model. The Abutment Angle Model utilises the panel and pillar geometry and the abutment loading parameters to calculate the abutment load associated with a longwall panel and thereby allows the chain pillar load to be calculated for the various stages of the chain pillar loading cycle associated with longwall operations (refer Figure 1).

In terms of the calculations, the required panel and pillar geometry are the solid (i.e. rib to rib) chain pillar width (\( w \)), roadway or entry width (\( w_e \)), cover depth (\( H \)) and longwall panel width (\( W \), being the maingate belt road to tailgate centre to centre distance i.e. \( W = \text{face width} + w_e \)).

The abutment loading parameters are essentially 1) the vertical pressure gradient (\( \gamma \)) which in ALTS is generally taken to be 0.025 MPa/m, 2) the abutment angle (\( \phi \)) and 3) the ratio between TG and MG Loading Conditions (i.e. \( \Delta TG: \Delta MG \)), where this ratio is calculated solely in terms of the additional measured abutment load subsequent to development.

It is important to note that; the abutment angle should not be considered as a physical reality (e.g. such as the angle of draw associated with surface subsidence measurements). The abutment angle and \( \Delta TG: \Delta MG \) are generally obtained (or back-calculated) based on field investigations with the abutment angle then used as a part of the Abutment Angle Model (i.e. a mathematical process) to estimate the portion of the overburden load not supported by the goaf (i.e. the side abutment load). The following discussion in combination with Figures 2 and 3 provides a more detailed explanation of the abutment loading parameters and how they are utilised in calculating the chain pillar load at the various stages of its loading cycle.

As LW 1 (refer Figure 2) laterally approaches and retreats past a chain pillar, the pillar goes through a dynamic loading cycle. This incorporates the development load (\( T \), calculated using tributary area concepts) plus the onset of a front abutment load as the longwall face approaches the pillar. The portion of abutment load carried by the chain pillar increases (up to a limit) as the longwall faceline continues to retreat outbye. Once the face retreats to a distance sufficiently removed from the chain pillar, static (i.e. side abutment or MG) loading conditions apply (refer Position c – Figure 1).
Figure 2: Abutment angle loading model

Subsequent to the full extraction of a longwall panel, the total abutment load is considered (as a reasonable approximation) to be evenly distributed on either side of the longwall panel centre line. Therefore the side abutment load \( A, \text{MN/m} \) is defined as that portion of the redistributed load associated with half the longwall panel width \( \frac{1}{2}W \) per metre of gateroad (as illustrated in Figure 2).

Dependent on the longwall panel width to cover depth ratio \( W/H \) and the size of the abutment angle \( \phi \), the abutment area may first intersect \( \frac{1}{2} \) longwall panel width or extend to the surface (refer Figure 2), which determines the equation utilised to calculate the side abutment load \( A \) as detailed on Figure 2.

At the MG Loading stage of the longwall extraction cycle, the side abutment load \( A_1 \) associated with LW 1 is distributed between the chain pillar and the solid coal within the unmined LW 2 (refer Figure 2). To estimate the portion of the side abutment load carried by the chain pillar (at MG Loading) use is made of the stress decay curve employed within ALPS, which is illustrated by Figure 3.

Figure 3: Abutment stress distribution function

\[ D = 5.13 \sqrt{H} \quad \text{(after Peng & Chang, 1981)} \]
In relation to Figure 3, D is referred to as the Abutment Influence Zone, which is based on field measurements analysed by Peng and Chiang (1984). They defined this zone as the distance from the panel edge that abutment stress increases could be detected. Field measurements associated with Australian studies (i.e. Colwell, 1998 and Colwell, 2006) are in general agreement, albeit with some notable exceptions.

Utilising the Abutment Angle Model, Abutment Stress Distribution Function as well as longwall panel and roadway centre to centre distances, the load carried by the chain pillar under MG Loading is estimated to be $T + RA$ (MN/m), where R is referred to as the Stress Reduction Factor and is calculated as follows:

$$R = 1 - \left[ \frac{(D - w - w_e)}{D} \right]^3$$

As LW 2 retreats (refer Figure 2), the chain pillars located between LW’s 1 and 2 go through a 2nd dynamic loading cycle. This incorporates the development load and the first side abutment load plus the onset (and therefore a portion) of a second front abutment load. Where the longwall panels are of equal width (i.e. $W_1 = W_2$) and the cover depth remains reasonably constant (therefore $A_1 = A_2$) then the ratio between Tailgate and Maingate Loading Conditions ($\Delta TG: \Delta MG$) can be used to estimate the TG Load such that:

$$TG \text{ Load (MN/m)} = T + \Delta TG: \Delta MG \cdot A_1$$

Once the second longwall face retreats to a distance sufficiently removed from the pillar then Double Goaf Loading Conditions apply and in theory this results in the chain pillars being subjected to the development load plus twice the side abutment load (i.e. $L = T + 2A$). Therefore in theory $\Delta TG: \Delta MG$ must lie between 1 and 2, however field investigations undertaken in Australia challenge the theory and therefore challenge how to best use the Abutment Angle Model as a part of an industry wide geotechnical design tool.

**SUMMARY OF FIELD INVESTIGATIONS**

To provide commentary on the appropriate use of the Abutment Angle Model for longwall tailgate design it is worth summarising the chain pillar monitoring studies reviewed by the author, which also provides some historical basis associated with the development, understanding and use of the Abutment Angle Model. There are essentially four sources of information:

- The research/field investigations associated with the ALPS research.
- The six chain pillar monitoring studies associated with Australian Coal Association Research Program (ACARP) Project C6036 entitled, “Chain Pillar Design – Calibration of ALPS” (Colwell, 1998).
- Thirteen chain pillar monitoring studies conducted in Australia prior to 1998, which were reviewed as a part of ACARP Project C6036.
- Subsequent to 1998, mine sites have provided the author with six chain pillar monitoring studies conducted by Strata Engineering Australia Pty Ltd (SEA) and three such studies undertaken by SCT Operations Pty Ltd (SCT).

The initial research in developing ALPS was undertaken by Mark and Bieniawski (1986) and further refined (Mark, 1990, 1992 and Mark et al, 1994) under the auspice of the United States Bureau of Mines (USBM). The initial research was carried out over five years with field measurements conducted at five separate mines (incorporating 16 longwall panels) to correlate an abutment angle ($\phi$) with measured abutment loadings.

The full side abutment load was successfully measured at six of the sites where the cover depths ranged from approximately 140 m to 230 m. It is understood that all stress cells were installed within the chain pillars with none located within adjacent longwall panels or solid blocks. The full side abutment load was estimated using the Stress Reduction Factor (R), while calculated abutment angles were 10.7°, 17.3°, 18.5°, 20.3°, 21.8° and 25.2°. The six sites analysed by Mark (1990) have W/H ratios ranging from approximately 1 to 1.9 with no apparent correlation between the abutment angle and cover depth, panel width or W/H.
Unfortunately TG Loading could only be measured at one site and it was found that the 
tailgate front abutment factor (\(F_t\), being the fraction of the tailgate side abutment reporting to 
the chain pillar at the T-junction) was 0.7. Therefore in terms of ALTS, this would equate to 
\(\Delta TG : \Delta MG\) of 1.7, where \(W_1 = W_2\). Based on the field investigations, Mark (1990) 
recommended generic values of \(\phi = 21^\circ\) and \(F_t = 0.7\) (i.e. \(\Delta TG : \Delta MG = 1.7\)) in assessing the 
abutment loads as a part of utilising ALPS for geotechnical design purposes.

By the mid-1990’s there was recognition in Australia of the need to develop a chain pillar 
design technique specifically related to roadway serviceability. In 1997 with ACARP and 
colliery support a research program (ACARP Project C6036, Chain Pillar Design – Calibration 
of ALPS) commenced to develop such a method. The starting point or basis of that research 
program was ALPS and therefore the chain pillar monitoring sites associated with the ALTS 
research were set up in a similar manner to that used in the development of ALPS. For 
example the goal of the ALPS field investigations (and therefore for ALTS) was to measure 
the abutment loads rather than the total loads carried by the chain pillars. Therefore it was 
only necessary to utilise stress cells that measured the change in vertical load during 
adjacent longwall extraction.

The chain pillar widths associated with the six instrumentation sites associated with ACARP 
Project C6036 varied from approximately 25 m to 40 m (rib to rib), while Figure 4 illustrates a 
proposed instrumentation layout for a 30 m wide pillar.

The stress cells were installed across the pillar from the cut-through at around mid-seam 
height and at a distance of approximately 10 m to 11 m from the cut-through ribline. In 
addition the cells were installed on the inbye side of the cut-through. Therefore if for any 
reason the wiring associated with the remote read-out was compromised, this allowed for the 
cells’ pressure gauges (at the mouth of hole) to be read up to and including Tailgate Loading 
(i.e. where the longwall face is in line with the stress cells).

As well as the stress cells within the pillar, three cells were also installed within the 2\(^{nd}\) 
adjacent longwall panel with a view to better defining the load distribution between the chain 
pillar and unmined panel at the MG Loading stage of the longwall extraction cycle and also to 
assess the correctness of the Stress Reduction Factor (R).
In addition to the array of stress cells; rib and roof extensometers were also utilised to assess the degree of movement and depth of softening within the pillar and roof. The stress cells immediately adjacent to the gateroad riblines often lost load as the longwall face approached and passed a site and in these instances the rib extensometers were particularly useful in providing corroborating evidence that the coal within the section of the pillar where the stress cell was located had in fact yielded under increased vertical load as opposed to a failure of the cell.

With respect to the six instrumentation sites, three were located in the Bowen Basin Coalfield (i.e. Central, Crinum and Kenmare), two collieries within the Newcastle Coalfield (i.e. Newstan and West Wallsend) and West Cliff Colliery in the Southern Coalfield. Table 1 summarises the panel and pillar geometry

**Table 1: Summary of panel and pillar geometry**

<table>
<thead>
<tr>
<th>Monitoring Site</th>
<th>H (m)</th>
<th>W1 (m)</th>
<th>W2 (m)</th>
<th>W1/H</th>
<th>h (m)</th>
<th>w (m)</th>
<th>we (m)</th>
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<tbody>
<tr>
<td>Central</td>
<td>265</td>
<td>200</td>
<td>230</td>
<td>0.75</td>
<td>2.5</td>
<td>39.9</td>
<td>5.1</td>
</tr>
<tr>
<td>Crinum</td>
<td>125</td>
<td>275</td>
<td>275</td>
<td>2.20</td>
<td>3.6</td>
<td>30.2</td>
<td>4.8</td>
</tr>
<tr>
<td>Kenmare</td>
<td>130</td>
<td>200</td>
<td>200</td>
<td>1.54</td>
<td>3.1</td>
<td>24.8</td>
<td>5.2</td>
</tr>
<tr>
<td>Newstan</td>
<td>180</td>
<td>130</td>
<td>130</td>
<td>0.72</td>
<td>3.2</td>
<td>26.0</td>
<td>5.0</td>
</tr>
<tr>
<td>West Cliff</td>
<td>475</td>
<td>200</td>
<td>200</td>
<td>0.42</td>
<td>2.5</td>
<td>37.2</td>
<td>4.8</td>
</tr>
<tr>
<td>West Wallsend</td>
<td>240</td>
<td>145</td>
<td>145</td>
<td>0.60</td>
<td>3.2</td>
<td>30.1</td>
<td>4.9</td>
</tr>
</tbody>
</table>

Although not published, the author became aware during the original ALTS research that in the United States some concern had been expressed that ‘ALPS does not work very well’ at deep cover with particularly strong ground conditions. The chain pillar monitoring studies (undertaken and reviewed as a part of ACARP Project C6036) associated with the deep cover South Coast (Illawarra) and Ellalong collieries identified the probable reason for that concern. Simply the TG Load is overestimated by using a generic abutment angle of 21°. The empirical analyses confirmed that to be the case.

However it was recognised that in most instances ALPS provided sound recommendations in relation to chain pillar sizing and was a proven chain pillar design tool such that prudent engineering judgment (based on compelling evidence) needed to be exercised before varying any of the loading parameters associated with the ALPS methodology. In the development of ALTS, the following criteria were applied to the calibration/adaptation process for Australian conditions:

- The ALPS abutment loading parameters utilised to estimate the TG Load (i.e. the design condition) should be maintained unless there is substantial evidence to the contrary.
- If changes to the abutment loading parameters are required, then to maintain the broad approach of the design methodology it is desirable to retain generic loading parameters.
- However where site specific corroborative information (both measured and anecdotal) is available, which indicates a significant departure from the ALPS abutment loading parameters then such changes will be made.

The tailgate loading data clearly indicated a generic ΔTG:ΔMG of 1.5 (rather 1.7) would be more suitable for most Australian collieries, while it was assessed that there was no benefit to be gained in altering the generic abutment angle of 21° except for some notable exceptions as previously alluded to. Subsequent to the original ALTS project direct industry support was given to complete the ALTS II project (refer Colwell et al, 2003), which significantly increased the database. The combined database (i.e. information collected in 1997/98 and 2000) represented 31 collieries involving some 140 data sets.

Based on the above research and an SEA chain pillar monitoring investigation in 2000 associated with Southern Colliery in the Bowen Basin Coalfield (confirming previous Central Colliery investigations both conducted and reviewed as a part of ACARP Project C6036), the
following abutment loading recommendations were provided to the Australian industry in the use of ALTS for tailgate design:

An Abutment Angle of 21° and $\Delta TG: \Delta MG$ of 1.5 (i.e. default values) should be used; with the only variance to these default values being advised for the following specific instances:

- South Coast (Illawarra) Collieries operating in the Bulli Seam where $H > 350$ m and Southland (formerly Ellalong) Colliery - the recommended values for $\phi$ and $\Delta TG: \Delta MG$ are 10° and 1.5. (note: this recommendation has also applied to Austar Mine)
- Central and Southern Collieries, Bowen Basin Coalfield - the recommended values for $\phi$ and $\Delta TG: \Delta MG$ are 26° and 1.6 (note: for future workings these values are applicable to lithology that is similar to that being experienced at each colliery).

The subsequent chain pillar monitoring studies conducted by SEA and SCT, which have been reviewed have since confirmed that the above recommendations are still the most practical way of estimating the chain pillar load associated with the TG Loading Condition (refer Position d - Figure 1) for all current Australian longwall operations and therefore the TG SF for chain pillars subject to double pass longwall extraction. These recommendations are still embedded in the current version of ALTS i.e. ALTS 2009 (refer Colwell and Frith, 2009).

**CASE STUDIES OF PARTICULAR NOTE**

Figure 5 plots $\Delta TG: \Delta MG$ against the Relative Face Position of the 2nd Adjacent Longwall face for the six chain pillar monitoring studies associated with ACARP C6036.

![Figure 5: $\Delta TG: \Delta MG$ versus 2nd LW face position](image)

As Figure 5 illustrates, both Maingate (i.e. the full side abutment load) and Tailgate Loading were successfully measured at all six sites. At three of the six sites Double Goaf (or at least close to Double Goaf) Loading was also measured prior to the cabling for remote read-out being destroyed. They were Central, Kenmare and Newstan collieries. Please note in relation to Central Colliery $W_2$ of 230 m is greater than $W_1$ of 200 m and therefore it is likely that the measured $\Delta TG: \Delta MG$ of 1.8 is slightly greater than had $W_2$ equaled $W_1$ as was the case for all other sites.

While the three Bowen Basin collieries (and for that matter even Newstan Colliery up to TG Loading) reasonably approximate the Abutment Angle Model utilising the generic ALTS abutment loading parameters, clearly West Wallsend and West Cliff Collieries do not. Also based on the Abutment Angle Model, in theory, the maximum measured change in vertical
load associated with double pass longwall extraction should not be greater than 2A and clearly West Wallsend and Newstan collieries significantly exceed the theoretical cut-off.

Figures 6 and 7 respectively display the measured stress profiles in relation to the extraction of Longwall Panels 16 and 17 associated with the West Wallsend Colliery chain pillar monitoring site. Figure 6 also displays the theoretical stress distribution associated with MG Loading based on an abutment angle of 21° and the stress distribution function displayed on Figure 3.

Subsequent to MG Loading there was significant concern that Cells 3 to 7 inclusive were malfunctioning with the measured/back-calculated abutment angle was only equal to 8.5°, which is based on the calculated MG Load being the integral under the stress profile with the LW 16 face position 242 m outbye of the site. However during the retreat of LW 17 (i.e. the tailgate loading phase – refer Figure 7) it became clear that such concern was not warranted and that an interesting loading phenomenon had occurred.

Figure 6: Maingate loading phase stress profiles – West Wallsend Colliery
As illustrated by Figure 7, the abutment load that had not reported to the chain pillar as part of MG Loading came into full force during tailgate loading. The integral of the stress curve associated with the LW 17 faceline 1m outbye of the cells (which closely approximates the TG Loading Condition) results in a $\Delta \text{TG}:\Delta \text{MG} = 3.79$. Based on $\phi$ and $\Delta \text{TG}:\Delta \text{MG}$ of 8.5° and 3.79 and the panel/pillar geometry (refer Table 1) the average chain pillar stress is 20.53 MPa resulting in a TG SF = 1.22. If the recommended generic ALTS abutment loading parameters of $\phi$ and $\Delta \text{TG}:\Delta \text{MG}$ of 21° and 1.5 had been used the calculated average chain pillar stress would be 20.12 MPa resulting in a TG SF = 1.24 i.e. a minor to negligible difference.

Colwell (1998) suggested that the nature of the overlying strata (i.e. a number of relatively thick sandstone and/or sandstone/conglomerate units within the overburden) in combination with the sub-critical nature of the panel width i.e. $W/H = 145 \text{ m}/240 \text{ m} = 0.6$), may result in the distribution of the side abutment load over a wider area rather than concentrating the side abutment load on the chain pillar and immediate rib line of the adjacent longwall panel during MG Loading as per the Abutment Angle Model. The stress cells within the adjacent longwall panel do indicate there is a gradual stress increase moving away from the solid rib (refer Figure 6).

Wold and Pala (1986) had made similar observations in relation to chain pillar monitoring studies conducted by CSIRO at Ellalong Colliery where they state, “The visual evidence of heavy abutment loads being distributed about the longwall block more broadly than might have been expected on theoretical grounds tended to be supported by the field measurements”.

As illustrated in Figure 5, the last successful stress cell measurements associated with Newstan Colliery (prior to the cabling being destroyed) occurred with the 2nd adjacent longwall face (LW 11) being approximately 71 m outbye of the line of stress cells. Given the shape of the curve and the panel width/cover depth ratio it is unlikely that full double goaf loading had been established at this point, however if one were to assume this to be double goaf loading then based on the measured change in load being equal to a theoretical $2A$, then a back-calculated abutment angle of 22.8° results.
All chain pillar monitoring sites simply represent one 2D slice of loading information associated with an entire tailgate and due to the costs involved, these means they are not routinely employed. However there are certain monitoring programs that have been successfully implemented to record the performance of various outcomes and strata behaviour associated with the entire longwall retreat. These are generally in relation to subsidence (i.e. survey lines), longwall face weighting (chock monitors) and roof extensometry devices regularly installed along the gateroad.

What all these types of regular monitoring exercises have clearly shown is that standard deviation (i.e. variance about the mean result) is ‘alive and well-developed’ in regards to underground coal mining strata mechanics. Therefore there is no reason to assume that the variance in chain pillar loading would be any different, if the monitoring of chain pillars along the full length of a panel were to be undertaken.

It was noted by Colwell (1998) that the tailgate performance at Newstan Colliery was quite variable and it was considered highly likely that a significant factor contributing to that variability was a changing TG Loading Condition probably similar to that, which had occurred at the neighboring West Wallsend Colliery. Both collieries carried out extraction in the West Borehole Seam with similar overburden lithologies and reduced panel widths as part of controlling periodic weighting associated with near-seam, thick massive conglomerate units.

Finally in relation to chain pillar monitoring studies; it is worth noting the results associated with the study conducted by SEA at Angus Place Colliery (within the Western Coalfield) in 2009/10 in the context of the findings associated with ACARP Project C6036. The chain pillar monitoring site was located adjacent to 20 c/t Maingate Panel 950 at a cover depth (H) of approximately 360 m, chain pillar width (w) of 43 m and adjacent longwall panel widths (W) of 287 m.

With respect to the Western Coalfield Colwell (1998) states, “It is assessed that given the wide range of mining environments there is insufficient (reliable) monitoring data in relation to the Western Coalfield collieries contained within the database (i.e. Angus Place, Clarence, Springvale and Ulan Collieries) so as to significantly modify the tailgate loading behaviour proposed by ALPS”. Therefore the recommended abutment loading parameters for the Western Coalfield collieries became \( \phi \) and \( \Delta TG: \Delta MG \) of 21° and 1.5 specifically to assess the TG Loading Condition in undertaking tailgate design using ALTS.

Over the years it was put to the author that the above recommendations overestimated the TG Loading Condition in relation to Angus Place and Springvale collieries, which would result in chain pillar widths greater than what certain industry personnel considered necessary based on other proposed abutment loading concepts.

The SEA chain pillar monitoring study conducted at Angus Place Colliery finally ‘put to bed the debate’ concerning ALTS’ suitable application in relation to the Western coalfield collieries. The resultant measured/back-calculated abutment loading parameters associated with the SEA investigations were \( \phi \) and \( \Delta TG: \Delta MG \) of 13.5° and 2.4, which would result in a TG Load of approximately 933.4 MN/m resulting in a TG SF = 1.07. If the ALTS recommended values of \( \phi = 21^\circ \) and \( \Delta TG: \Delta MG = 1.5 \) are used then the calculated TG Load is 932.8 MN/m resulting in a TG SF = 1.07 i.e. no difference to the 2nd decimal place.

As previously discussed with respect to the quite variable Newstan Colliery tailgate performance; it is highly likely that individual chain pillars (or various 2D slices of a chain pillar) along the full length of the Angus Place 950 Panel would be subject to a wide range of MG Loading abutment angles and associated \( \Delta TG: \Delta MG \), while it is likely that the resultant TG Load in most cases would have closely approximated the use of the Abutment Angle Model utilising the generic abutment loading parameters \( \phi \) and \( \Delta TG: \Delta MG \) of 21° and 1.5, that had been recommended by Colwell (1998).

More recently Hill et al (2015) provided a predictive formula to estimate the abutment angle based on cover depth, the chain pillar width and longwall panel width to cover depth ratio. However it is important to note that the formula predicts the MG Loading abutment angle and no guidance or formula is provided with respect to \( \Delta TG: \Delta MG \). In fact in relation to \( \Delta TG: \Delta MG \)
Hill et al (2015) state, “This factor was also found to be highly mine specific and not to relate strongly to the geometrical factors influencing the abutment angle.” Without a credible predictive capability for $\Delta$TG:$\Delta$MG, which relates directly to the measured MG Loading abutment angle, then having only a predictive capability for the MG Loading abutment angle is not even ’half the story’ with respect to a credible estimate for the TG Load.

CONCLUSIONS

Given that the chain pillar minimum width/height (w/h) ratio associated with the vast bulk of Australian longwall operations is > 8 and the effective w/h (based on the UNSW Rectangular Pillar Strength Equation, refer Galvin et al, 1999) is > 15 this negates the possibility of inner core pillar failure. Therefore the principal use of the Abutment Angle Model for practical purposes is for the design of chain pillars as a part of an overall tailgate design strategy/methodology such as ALTS.

While the Abutment Angle Model also finds application as part of the Analysis and Design of Rib Support (ADRS) Design Methodology (Colwell, 2006) and assessing induced horizontal stress increases in adjacent roadways due to Poisson’s Effect, nonetheless its principal purpose is to estimate the chain pillar TG Load adjacent to the travelling tailgate intersection with the 2nd adjacent longwall face (refer Position d, Figure 1) being the design condition associated with ALTS.

Based on the field investigations, the variability with respect to the abutment loading parameters is clearly more pronounced for collieries associated with the NSW coalfields as opposed to those located in the Bowen Basin, which tend to more closely and consistently approximate the Abutment Angle Model. The findings associated with ACARP Project C6036 and subsequent ALTS research clearly recognised and effectively accounted for this in terms of the recommendations provided.

The ALTS recommended abutment loading parameters, which have been clearly explained in all publications associated with ALTS and during the numerous training courses provided by the author to industry personnel over the years, provide the most effective means of estimating the TG Load associated with current Australian longwall operations and therefore the need for any further chain pillar monitoring investigations is extremely limited and in most cases would be only of an academic interest.

REFERENCES

Wold, M B and Pala, J, 1986. Three-dimensional stress changes in pillars during longwall retreat at Ellalong Colliery. CSIRO, Australia, Division of Geomechanics. Coal Mining Report No. 65. 44p
COAL PILLAR DESIGN WHEN CONSIDERED A REINFORCEMENT PROBLEM RATHER THAN A SUSPENSION PROBLEM

Russell Frith¹, Guy Reed

ABSTRACT: Current coal pillar design is the epitome of suspension design. In principle, this is seemingly no different from early roadway roof support design. However, for the most part, roadway roof stabilisation has progressed to reinforcement, whereby the roof strata is assisted in supporting itself. Suspension and reinforcement are fundamentally different and, importantly, lead to substantially different requirements in terms of roof support hardware characteristics and their application. This paper presents a prototype coal pillar and overburden system representation where reinforcement, rather than suspension, of the overburden is the stabilising mechanism via the action of in situ horizontal stresses within the overburden, the suspension problem potentially being an exception rather than the rule, as is also the case in roadway roof stability. Established principles relating to roadway roof reinforcement can potentially be applied to coal pillar design under this representation. The merit of this assertion is evaluated according to documented failed pillar cases in a range of mining applications and industries found in a series of published databases. Based on the various findings, a series of coal pillar system design considerations and suggestions for bord and pillar type mine workings are provided. This potentially allows a more flexible and informed approach to coal pillar sizing within workable mining layouts, as compared with common industry practices of a single design Factor of Safety (FoS) under defined overburden dead-loading to the exclusion of other potentially relevant overburden stabilising influences.

INTRODUCTION

The simplest model for coal pillar loading consists of an unstable overburden to the surface, known as Tributary Area Theory (TAT), overburden stability then being entirely controlled by the load-bearing ability of the coal pillars formed in the workings (Figure 1). For bord and pillar type mining design purposes, the TAT model to the surface has been and can be modified (by the application of either pressure-arch concepts or by considering the sub- or super-critical nature of the overburden at the surface) to modify pillar loading magnitudes, it is still generally true to state that the stability of coal pillars is evaluated via a defined unstable section of overburden imparting dead loads onto the coal pillars beneath. The level of confidence in the design remaining stable is then determined according to the design Factor of Safety (FoS) over and above the assumed coal pillar strength(s).

Figure 1: Tributary Area Theory (TAT) loading arrangement for coal pillars

Since the Coalbrook disaster in 1960, the basic model of full TAT to surface has been applied in empirical studies attempting to define the strength of coal pillars by back-analysing failed cases (e.g. Salamon and Munro 1967). Figure 2 shows how pillar loading can be modified

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according to panel width to cover depth considerations (\(W/H\) ratio) as part of what may be termed as partial TAT [Mark et al 2010]. In both cases, vertical dead-loading of the overburden onto the coal pillars is the key pillar design assumption.

![Diagram of pillar designs](image)

(a) Super-critical panel  (b) Sub-critical panel

**Figure 2: Abutment angle concept used to estimate loads in ARMPS [Mark et al 2010]**

Van Der Merwe 2006 makes the following statement in relation to what occurred in the immediate aftermath of Coalbrook: “The attention was focused on pillar strength research, very little attention initially being paid to overburden strength. This is not an unreasonable approach: if the pillars are strong enough to support the overburden, it doesn’t matter how weak the overburden is—failure cannot occur. This is especially true if the [TAT] is used to determine pillar load because TAT predicts the maximum load on the pillar”.

The fact that this statement was made as recently as 2006, over 40 years after Coalbrook, is taken to be evidence that the TAT pillar loading model has persisted, whether it be full TAT to the surface or a modified/partial version.

At the 35th ICGCM, Reed, McTyer, and Frith 2016 posed a question, asking whether it was possible for coal pillar research to follow what had already occurred in roadway roof control research. Roadway roof control was initially founded on the belief that roadway roof support needed to be designed to hold in place an otherwise critically unstable roof mass, using suspension roof support [Figure 3(a)]. However, this was eventually superseded by the prevailing concept that roadway roof stability could be far more efficient and reliable by retaining some or all of the self-supporting ability of the roof strata via reinforcement using roof bolting and longer cables and tendons (Figure 3(b)). The reinforcing approach considers the competence of the roof mass (as given by the Coal Mine Roof Rating or CMRR for example), the horizontal stresses acting across the roof, the width of the roadway and the installed roof support in formulating design outcomes. The roadway roof stability design problem was forever changed from the simple and often far too simplistic assumption of “dead-load” suspension when the problem became one of roof reinforcement. Frith and Colwell 2011 outline details of the various problems and potential risks of continuing to apply a dead-load suspension approach to roadway roof support design in reinforcing design situations.

![Diagram of roadway roof control](image)

**Figure 3: Schematic illustrations of suspension and reinforcing problem representations for roadway roof control**

This paper debates the application of full or modified TAT dead-load pillar loading to bord and pillar coal pillar design nearly 60 years after Coalbrook. Figure 1 shows the suspension arrangement for coal pillars. The similarity with Figure 3(a) for roadway roof support is evident; the only difference being that the unstable strata mass is held in place
by roof bolts anchored into stable overlying strata, as compared to a coal pillar being founded on the floor of the roadway.

In stark contrast, Figure 4 outlines a suggested reinforcing problem representation for coal pillars similar to that shown in Figure 3(b) for the roof of a mine roadway. In the reinforcing coal pillar design representation, the horizontal stress acting within the overburden, the competence of the overburden in terms of its self-supporting ability across the panel, and the panel width are all brought into the problem representation. These are directly analogous to the horizontal stress in the roof of a roadway, the competence of the roof strata (e.g. the CMRR), and the roadway width, respectively. Each is a primary variable in the reinforcing roadway roof stability problem.

![Image](image_url)

**Figure 4: Schematic illustration of a reinforcing problem representation for coal pillar design**

This then leads to the fundamental question as to whether, albeit with many years of hindsight, the mechanics of coal pillar design are comparable to that of reinforcing roadway roof support. Do coal pillars control the overburden through reinforcement rather than suspension? Do pillars work to allow the overburden to stabilise itself, rather than simply support the overburden?

The paper seeks to demonstrate that the overburden reinforcement scenario is the more likely answer in many instances with some specific exceptions and offers views on the implications on bord and pillar layout design involving coal pillars.

**JUSTIFICATION FOR A REINFORCING APPROACH TO COAL PILLAR DESIGN**

In addressing whether the coal pillar design problem is one of suspension or reinforcement, it must be determined which becomes unstable first: the overburden to surface or the pillar? This question is derived by considering suspension design for roadway roof support where by definition, the installed roof support must remain load-bearing well after the roof strata has failed and become critically unstable, a roof collapse being solely dictated by the structural state and associated load-bearing capacity of the installed roof support.

The earlier quotation from Van Der Merwe 2006 implies that as long as the overburden becomes critically unstable before a correctly designed coal pillar reaches its maximum load-bearing capacity, it does not matter how weak the overburden is at face value this makes perfect sense, but leaves one critical issue unanswered: “Will in fact the overburden become critically unstable before the coal pillar goes post-peak”?

If the answer to the question is generally “no”, it must logically be concluded that the manner by which the suspension approach to pillar design as outlined in Figures 1 and/or 2, whereby an unstable amount of overburden is controlled by coal pillars prior to their peak strength being reached, is a worst, a flawed and at best, a limiting view. Therefore, how and why the overburden becomes critically unstable relative to the coal pillar becoming unstable becomes of significant interest.
Useful thought-experiments

Before presenting detailed technical arguments, it is useful to consider a number of basic “thought-experiments” whereby the significance of the horizontal stress in the overburden can be justified as a problem variable, both conceptually and numerically.

The Ground Reaction Curve (GRC) concept (Figure 5) was originally developed in the early 1960’s to assist tunnellers ensure that permanent, and often, stiff permanent tunnel linings were not damaged by excessive ground strains. This has since been applied by others to coal mining problems, such as tailgate standing support design and longwall shield design.

Figure 5: Generic Ground Reaction Curve (GRC) representation

Figure 5 shows that a significant element of a GRC is the installation of ground support following excavation; however in this instance it is only the ground curve that is of interest. The ground curve (ABCD contains both a section of negative slope (ABC) whereby the strata is losing its natural stability due to increasing vertical movement and a section of positive slope (CD) whereby natural stability is effectively lost with self-weight or dead-loading then dominating the stability problem.

A series of simple thought-experiments relating to super-critical extractions (i.e. W/H > 1) will now be used to demonstrate the direct influence of key geotechnical parameters on initial overburden caveability. Figure 6 presents a simple situation whereby the overburden consists solely of massive sandstone to the surface with no vertical joints. Under this scenario, it is self-evident that the overburden will retain its stability across the extraction span with the ground curve rapidly reducing to zero stress (i.e. self-supporting) following extraction.

Figure 7 is geometrically the same as Figure 6 but the overburden now solely consists of laminated material, albeit still with no vertical joints. Under this scenario, the overburden initially flexes via vertical downwards movement, but eventually exceeds some undefined “critical” level of overburden movement which marks the onset of mass instability back to full overburden collapse (under tributary area loading in coal pillar design terminology).
Figure 6: Thought-experiment and ground curve: massive overburden with no vertical joints

Figure 7: Thought-experiment and ground curve: Laminated overburden with no vertical joints

Figure 8 contains the massive and laminated overburden representations used in Figures 6 and 7, but introduces both vertical joints and horizontal stress.

Vertical jointing is almost always present in coal measures strata sequences (although spacings, orientations and persistence vary) and is typically characterised by zero cohesion and a friction angle that varies according to surface conditions along the joint. Under such joint conditions, no vertical shear resistance can be developed along the joint without the influence of a normal confining stress (horizontal stress in this case). Therefore irrespective of the overburden type, without the presence of horizontal stress vertical jointing results in a
ground curve as shown in Figure 9, namely the overburden is an unstable detached block from the outset.

*Figure 8: Thought-experiment representations: Massive and laminated overburdens including vertical joints and horizontal stress*

Once horizontal stress is included into the representation, it allows the varying influence of overburden lithology and vertical shear along vertical joints to be combined, resulting in a range of possible ground curve outcomes as generally illustrated in Figure 10. To provide context as to what may be a significant level of horizontal stress in this regard, assuming a joint friction angle of 45° only 2.5 MPa of horizontal stress is required for limit equilibrium in terms of shear slip along vertical joints at the extremities of a 200 m wide extraction span. This is a relatively low level of horizontal stress in general coal mining terms.

*Figure 9: Ground curve: Vertical joints present without horizontal stress*
In general terms, for any given extraction span, initial caveability is directly influenced by both the structural competence of the overburden in terms of the presence or absence of massive strata units etc. and also the level of horizontal stress acting within the overburden which defines the level of stabilising influence along vertical joints. The important point to make is that in combination they can result in two distinctly different caving mechanisms, namely:

- delamination and incremental overburden collapse whilst ever vertical joints do not undergo vertical shear slip, as opposed to;
- a “plug” type collapse of otherwise intact overburden material due to vertical shear slip along vertical joints.

The point of these thought-experiments and simple calculations is to justify that it is almost certainly overburden condition above coal pillars, which dictate their importance in maintaining the overall stability of mine workings. More to the point, the full TAT representation of Figure 1 is clearly only one possible pillar loading and overburden condition scenario and as will be now further argued, it is potentially quite uncommon for the peak-strength of the coal pillar solely controlling whether a pillar collapse (low w/h) or pillar creep (higher w/h) occurs or not.

**Which generally fails first—overburden or coal pillars?**

The starting point for an assessment of which is more likely to fail first - overburden or coal pillar- is again found in the (GRC) concept (Figure 5). The GRC concept was originally developed around the same time as the Coalbrook disaster to assist tunnellers to ensure that permanent, and often, stiff permanent tunnel linings were not damaged by excessive ground strains. Others have since applied this to coal mining problems, such as tailgate standing support design and longwall shield design.

Figure 5 implies that a significant element of GRC is in the installation timing of different types of ground support following excavation in that:

1. They should be installed before the surrounding strata becomes critically unstable. In reality, the surrounding strata becoming critically unstable is the onset of the suspension design problem, this being where the ground curve starts to rise with ever-increasing strata displacement.
2. They should be installed sufficiently early so that the ground and support loading curves coincide at some point. This is the point that overall equilibrium or stability is achieved.

It is suggested that applying the GRC concept to coal pillar design is a perfect analogy to attempting to protect a permanent tunnel lining from excessive ground strains. The reinforcement design problem for a permanent civil excavation for example is about...
controlling ground movements within acceptable limits, particularly if a key supporting structure could be overloaded as a direct result of being installed too early. However, two major differences are apparent when applying the GRC concept to coal pillars rather than excavation support, namely:

3. The pillars are already installed at the time of excavation (i.e. their installation cannot be delayed).
4. They are inevitably pre-loaded to a pre-determined level by the action of the in situ vertical stress, therefore their maximum elastic straining ability post-mining is inevitably reduced as cover depth increases (all other factors being equal).

Defining the initiation point for overburden instability

Considering the issue of the overburden first, the relevant question is how much vertical movement is required for the overburden to become critically unstable to surface at a super-critical panel width? (See Scenario A in Figure 2). For the purpose of demonstration, a panel width range between 150 and 200 m will be considered, these generally resulting in a super-critical mining geometry at relatively shallow (<150 m depth) bord and pillar mining (W/H>1).

Two sources of guidance will be used in addressing this question: (a) surface subsidence data with respect to the transition from sub-critical to super-critical surface behaviour (as illustrated in Figure 2), and (b) overburden extensometry data relating to measured overburden movements following the extraction of longwall panels in the same panel width range.

Figure 11 shows the standard $S_{max}/T$ vs $W/H$ representation for a series of varying width longwall panels in the Newcastle Coalfield, the cover depth ranging between 70 and 150 m [Ditton and Frith 2003]. This is also a common bord and pillar mining cover depth range. The data indicates that the onset of full overburden instability and associated collapse at the surface commences at an $S_{max}/T$ value of around 0.1, the mid-point of the transition to super-critical being 0.25 to 0.3, and full collapse at surface at a value in the order of 0.5. For an assumed mining height of 2 m, these represent vertical overburden movements at the surface of 200, 600, and 1000 mm respectively. In other words, overburden collapse at the surface does not typically commence under this scenario before 200 mm of vertical movement and is only reliably complete by 1000 mm. It is accepted that these values change according to varying extraction height and may also be influenced by overburden characteristics and goaf bulking behind a longwall. However, they are a useful starting point for this discussion.

Also of interest is the amount of vertical movement in the immediate overburden above the mine workings that is required to initiate overburden collapse across the full panel width, this section of strata directly interacting with the coal pillars that are left in place.

Figure 12 shows overburden movement isopachs in vertical section behind a longwall face as a function of both distance into the overburden above the working horizon and distance behind the face [Mills and O’Grady 1998]. While the figures themselves are not particularly clear, statements from the paper are worth re-quoting herein.
Figure 11: Measured $S_{\text{max}}$ values analysed according to extraction height ($T$), panel width ($W$) and cover depth ($H$) for depths ranging from 70 to 150 m [Ditton and Frith 2003]

Figure 12: Surface extensometry data, LW’s 4 and 5, Clarence colliery [Mills and O’Grady 1998]

The following is in relation to Figure 12(a) for a 150-m-wide longwall panel: “the 200 mm contour represents the line below which, downward movements accelerate rapidly. The transition between relatively small displacements (<200 mm) and much larger displacements occurs within a relatively narrow zone.”
Regarding Figure 12(b), Mills and O’Grady (1998) state that for a 200 m wide longwall panel: “immediately below the 200 mm contour, the rate of ground separation increases rapidly as indicated by the close spacing of the 500 mm and 1 m contours”.

In both instances, Mills and O’Grady (1998) are seemingly stating that the onset of rapid overburden movement following longwall extraction occurs at an overburden displacement magnitude in the order of 200 mm.

Independent and fundamentally different data sets relating to instability at the surface and within the overburden following longwall extraction, may cause minimum overburden movements prior to the onset of full overburden collapse in the order of 200 mm for panel widths in the range 150 to 200 m. This is the first requirement in determining whether the coal pillar or the overburden fails first when coal pillars are left in place between stable barriers.

**Condition of coal pillar at peak loading**

In terms of the coal pillar, the relevant consideration is the vertical compression at the point that it reaches its maximum or peak load-bearing ability, this being commonly termed as “pillar strength”. Figure 13 provides stress-strain curves relating to the laboratory testing of coal samples according to varying $w/h$ ratio [Das 1986]. From this data, it is estimated that for a $w/h$ range of 1 to approximately 5, the vertical strain at peak-loading varies from 1% to 2%. Extrapolating this to a coal pillar scale means that for a 2 m high pillar, the total vertical compression at peak loading varies between 20 and 40 mm. However, as discussed by Galvin, caution needs to be used when applying laboratory test data to in situ pillar behaviour; therefore, some form of in situ coal pillar testing data would be valuable to provide real-world insight into this issue.

![Figure 13: Stress-strain behaviour of coal for varying width to height ($w/h$) ratio [Das 1986]](image)

Figures 14 and 15 contain stress-strain data for larger coal “pillars” tested in situ as opposed to laboratory specimens [Van Heerden 1975]. In the two cases for two different $w/h$ ratios, the compressive strain at peak-loading is <1% (i.e. $10^{-3}$). In the case of a 2 m high coal pillar this equates to <20 mm of vertical compression at its maximum strength.
Figure 14: Stress-strain behaviour for in situ coal pillar testing [Van Heerden 1975]

Figure 15: Stress-strain behaviour for in situ coal pillar testing [Van Heerden 1975]

Figure 16 relates to a different method of in situ coal pillar testing. In this test, an approximately 1.8 m-high coal pillar is incrementally reduced in size and the applied pillar load and vertical compression responses are measured [Skelly et al 1975]. Recognising that the coal pillar is pre-stressed before it is reduced in size due to the in situ pre-mining vertical stress, an additional 2.54 mm of compression is recorded when taking the pillar from its in situ loading under the action of vertical stress 16.5 MPa (2,400 psi), to its measured peak load of around 2900 psi (20 MPa). This equates to 2.54 mm of pillar compression for a 3.5 MPa (500 psi increase in average pillar stress. Extrapolating this value to the complete elastic stress-strain range of the pillar from 0 to 20 MPa, results in total vertical compression of the pillar at peak loading of only 14.7 mm for the 1.8 m high pillar in the test case. For a 2 m-high pillar, this would increase to 16 mm, all other factors being equal.
Figure 16: Stress-deformation curve for test pillar “a” [Skelly et al 1975]

In situ coal pillar testing data from both South Africa and the USA indicate that for a 2 m high coal pillar, vertical pillar compression at peak-loading or maximum strength would be in the order of 20 mm. This is of similar magnitude as that inferred from the Das 1986 lab-testing data of 20 to 40 mm, according to a varying w/h ratio of between 1 and up to 5.

So which fails first—overburden or coal pillar?

Even making an allowance for the elastic compression of an immediate stone roof and floor above and below coal pillars, the preceding general analysis of overburden movements above total extraction panels and vertical strains in coal at peak-strength, leads to the inevitable conclusion that it is more likely than not that coal pillars will exceed their peak strength (and so “fail”) before the overburden becomes in a critically unstable state to the surface.

This general conclusion is well justified in the various reports relating to Coalbrook in that (a) surface subsidence cracks above the area of the mine that was eventually to collapse so catastrophically, were identified days before the major event (NB subsidence cracks at surface generally require hundreds of millimetres of vertical subsidence before they appear, not tens of mm), (b) various coal pillars were observed to be spalling and splitting well prior to the main collapse (which is inconsistent with the retention of an elastic pre-peak pillar state leading up to the collapse) and (c) micro-seismic events were heard and measured as part of the main collapse, a common source of micro-seismic and indeed seismic events being stress-driven shear slip along pre-existing planes of weakness (such as the vertical joints in Figure 8), rather than the compressive failure of soft material as in a coal pillar.

One case example on its own obviously does not prove that all failed pillar cases in the various coal industry databases conform to this same scenario. Weaker and less stiff roof and/or floor measures, thick soft clay bands within the coal seam, very low horizontal stress environments in mountainous terrain or in proximity to highwalls and/or the presence of mid-angled structures in the overburden above a collapsed area, would all tend to change the conditions at the point of overburden collapse towards one whereby the coal pillar may be the controlling influence, as analysed previously. However, the various observations leading up to the Coalbrook collapse certainly support the idea that the overburden failing before the coal pillar can be eliminated as a universal truth. If nothing else, this means that any coal pillar strength equations that have been empirically-derived from databases of failed pillar cases,
will only reasonably reflect the intact strength of coal pillars if all or the majority of the failed cases occurred as a direct and immediate consequence of the coal pillars first exceeding their intact strength, the Coalbrook collapse now being strongly argued to not fit this description.

This finding, in particular as it relates to Coalbrook, is seemingly quite profound and raises a question-mark as to the reliability of almost 60 years of coal pillar strength research that has utilised failed pillar cases in back-calculating intact coal pillar strengths at failure and so developing coal pillar strength equations. However, it is only based on one technical argument, namely that coal pillars can reach their peak-loading condition well before the overburden has displaced sufficiently to become critically unstable. Therefore, further supporting evidence has been sought.

Other real-world examples worth considering further

Three significant case history examples have been identified that either (1) seemingly further back the assertion that the rigid application of full TAT loading to surface (Figure 1) when back-analysing failed pillar cases contains limitations that may only manifest when the resultant pillar strength equations are used in practice, and/or (2) indicate that a key piece of the design problem has been overlooked and that we may do well to now include it. Figure 17 shows the combined Australian and South African databases of failed coal pillars, including a series of Highwall Mining (HWM) pillar failures that were back-analysed using the previously-developed pillar strength equations of the University of New South Wales Pillar Design Procedure (UNSW PDP) [Hill 2005]. The area of interest (marked by a red ellipse) contains a number of HWM pillar failures with UNSW PDP FoS values ranging from 1.7 to as high as 2.4, these all being greater than, significantly so in most cases, the range of FoS values for the failed non-HWM cases that define the majority of the two databases.

It could perhaps be argued that these HWM pillar failures are due to low pillar w/h ratios having reduced pillar strengths as compared to the predicted strengths, due to the influence of localised geological structures. In fact, this influence is commonly argued by others, although at first glance it isn’t immediately obvious from Figure 17, as there are a number of non-HWM failed cases with similarly low w/h ratios (which would have also been square/rectangular underground pillars rather than continuous HWM pillars) that do not generally mirror those of HWM. Perhaps the HWM pillar failures that stand out from the general underground mining examples have a different causation entirely.

![Figure 17: Database of pillar collapses–width to height ratio vs. FoS [Hill 2005]](image)

Forming coal pillars next to an open cut highwall represents a specific and not particularly common scenario, whereby the self-supporting ability and stiffness of the overburden above the pillars is inevitably reduced due to the presence of a zero friction vertical discontinuity in the overburden with no horizontal stress acting across it, namely the highwall. If, as previously argued via the various thought-experiments and calculations, the overburden needs to overcome the stabilising influence of horizontal stress acting across vertical joints before it enters a critically unstable state, then being located adjacent to an open cut highwall is an obvious situation where this influence would logically be reduced as compared to further
into an underground mine. In other words, the HWM situation may more closely resemble that of Figure 1, as compared to that of underground mining. Therefore, if the UNSW PDP pillar strength equations as used by Hill 2005 contain a limitation that is linked back to their derivation, its presence and significance should logically show up in any case examples that significantly deviate from the majority within the founding database. To have five failed examples in the FoS range of 1.7 to 2.4 in such a small database as that of Australian HWM, strongly suggests that the coal pillar strength equations used in determining the FoS values in Figure 17, are substantially overstating coal pillar strength. In this regard, it is interesting to note that UNSW are currently undertaking a joint project with CSIRO to re-evaluate their pillar strength equations for low-width HWM pillars, presumably as a direct result of the Australian and other failed HWM examples.

The other notable aspect of Figure 17 is that, for \( w/h \) values >4 to 4.5, there is only one outlier collapsed failed case at a \( w/h \) of almost 8, which, according to Colwell 2010, is more likely to be a floor failure than a core pillar collapse based on his personal knowledge of the case history. As such, its relevance in terms of the discussion on whether the pillar or overburden fails first is judged to be minimal, as without definitive knowledge on the type of instability involved it cannot be included. The same trend is also evident in the ARMPS failed pillar database (Figure 18), where, in terms of core pillar failures there are no examples with \( w/h \) values >3 [NIOSH]. This leads the discussion into pillar squeezes or creeps and what the relevant case histories might be able to tell us about the general validity of pillar strength equations at higher \( w/h \) ratios.

![Figure 18: ARMPS SF vs pillar w/h ratio for pillar collapses and other case histories](image)

\[ \text{Figure 18: ARMPS SF vs pillar } w/h \text{ ratio for pillar collapses and other case histories} \text{ [NIOSH]} \]

Ditton and Frith 2003 analysed the relationship between surface settlements above longwall chain pillars (under Double Goaf (DG) loading with extraction panels on both sides) that could not be explained by the superimposition of the two individual subsidence troughs on either side of the chain pillar. The resulting levels of surface lowering were plotted against the FoS of the chain pillar under DG loading as determined using the UNSW PDP pillar strength equation for \( w/h \) values >5. A relationship between the two parameters was found as shown in Figure 19.
Figure 19. Surface lowering above a chain pillar under DG loading v chain pillar FoS using UNSW PDP strength equations [Ditton and Frith 2003]

This pillar loading scenario is probably quite unique in that due to the presence of extracted longwall panels on either side of the chain pillars, any stabilising influence of horizontal stress within the overburden should have been substantially reduced as compared to standing coal pillars in a bord and pillar type layout. Despite the extreme extraction geometry on either side of the pillars as compared to mine roadways, this scenario may in fact be more consistent with that of full TAT pillar loading and so provide further interesting insights. Surface lowering in above a chain pillar with longwall goaf on both sides is inevitably also influenced by the compression of the roof and floor strata, as well as that of the chain pillar itself. However, as the chain pillar controls the vertical stresses that are being applied to both the roof and floor strata, and as the coal has a Young’s Modulus that is generally substantially lower than that of stone, it is perhaps not unreasonable to examine the problem, at least in the initial instance, using this data set.

What is most interesting is that the general relationship in Figure 19 indicates that the magnitude of surface lowering starts to exponentially increase for chain pillar FoS values below the range of 2 to 2.5. The chain pillars in this database are all of a high w/h ratio as a result of Australian longwall mining practice, therefore coal pillar collapse via failure of the pillar core cannot be attributed as the cause. Therefore, for the onset of what appears to be non-elastic chain pillar compression effects at FoS values in the range of 2 to 2.5, one can infer that the pillar strength equations being used may be significantly overstating the intact or elastic peak-strength of the coal pillar. This is essentially no different from what has been concluded in relation to the Australian HWM cases in Figure 17.

Having analysed two different Australian databases that contain what appear to be anomalous pillar stability outcomes with respect to either pillar strength, there is perhaps a credible argument that the pillar strength equations used in both examples tend to overestimate the true intact strength of coal pillars across a wide range of w/h ratios.

Where all of this ultimately leads to is the statement that mine layout design for bord and pillar type mining may benefit by dispensing with the idea that the overburden can be stabilised at a prescribed level of design confidence by simply assigning an FoS (or SF) to the coal pillars in isolation from the natural stability of the overburden. Furthermore, if coal pillars do indeed commonly exceed their peak strength well before the overburden becomes critically unstable, their role in preventing overburden collapse cannot be linked to their peak strength, nor therefore by definition, the traditional pillar design FoS of pillar strength/pillar load. In other words, coal pillar FoS under vertical loading may be no more than a useful surrogate for some other, more relevant consideration.

This leads to the inevitable question as to how else can coal pillars act to stabilise the overburden above either bord and pillar or partial extraction mine layouts, whereby retaining overburden stability during and post-mining is almost always the critical layout design requirement? The only credible possibility is that of overburden “reinforcement” with the coal pillars assisting the overburden support itself; in the same way that reinforcing roof support
acts to assist the roof strata above a mine roadway to support itself. This however is another topic for another time.

**Another quick look at Coalbrook**

Returning to Coalbrook, it is interesting to re-consider the sequence of events leading up to the disaster as outlined by Van Der Merwe 2006. Two comments, in particular, are intriguing:

1. The main collapse on January 21, 1960, was preceded by a smaller collapse on December 28, 1959, in an experimental area where top coaling had been undertaken.
2. Mine management concluded, incorrectly as it turned out, that “the weight had come off” following the first two collapses on January 21, based on the occurrence of surface subsidence. They therefore assumed that the remaining areas were safe.

If one applies the full TAT pillar loading model shown in Figure 1 it is difficult to understand how such a large area of inadequately sized coal pillars (as evidence by their eventual collapse) could stand over years without incident, yet a pillar collapse in one, small area of only 6 hectares, triggered at least three major collapses in rapid succession totalling over 324 hectares, less than 1 month later. However, if one applies the overburden reinforcement model shown in Figure 4, the focus becomes one of overburden and coal pillars working in tandem to retain horizontal stress within the overburden. With this model in mind, the events at Coalbrook can be explained quite differently.

The smaller pillar collapse in December 1959 within the trial area of top coaling, would have inevitably caused a substantial reduction in horizontal stress within the overburden above nearby surrounding mine workings that were still standing. Similar to high magnitude guttering or cutter roof in the roof of a mine roadway, with the stabilising contribution of horizontal stress being lost above part of the overall pillar system remote from any barriers, the stability of the entire overburden would have inevitably been put at risk. As it turned out, a series of major collapses occurred not long afterwards.

It is interesting to dwell on the coal pillar strength research that occurred in South Africa following Coalbrook. The most intriguing issue are the numerous changes made to coal pillar strength equations as other collapsed cases have been considered from different coalfields. Currently this process is still ongoing, with new formulae being developed in response to new pillar failures. It is still continuing in Australia today with the UNSW highwall mining pillar strength formula [Mo et al. 2017]. Therefore, it could perhaps be argued that the process has now got to a point where selecting the appropriate pillar strength equation for design purposes is as much engineering judgement as assigning representative values to any other geotechnical parameter. This is hardly acceptable in a mine design discipline that carries the safety and business consequences associated with coal pillar failures.

With this in mind, it is instructive to further reflect on the coal pillar strengths equation developed after Coalbrook. Equation 1 is based on the in situ testing of large coal specimens with no direct link to failed cases and/or the assumption of full TAT [Bieniawski 1968] and it provides lower pillar strength values as compared with the statistically-derived Equation 2 and those of the UNSW PDP which use the assumption of full TAT as per Figure 1. However Equation 3 is the recently published strength equation for highwall mining coal pillars [Mo et al. 2017], highwall mining being one scenario whereby the stabilising influence of horizontal stress on the overburden can logically be minimised, which is very close numerically to that of Equation 1.

$$\sigma_p = 2.76 + 1.52 \frac{w}{h}$$  \hspace{1cm} (1)

$$\sigma_p = 7.176 \left( w^{0.46} / h^{0.66} \right)$$  \hspace{1cm} (2)

$$\sigma_p = 4.66 \left( 0.56 + 0.44 \frac{w}{h} \right) \text{ or } 2.61 + 2.05 \frac{w}{h}$$  \hspace{1cm} (3)

The conundrum that emanates from Equations (2) and (3) is that they suggest that a square pillar with a \( w/h \) of say 2 [Equation 2] is actually stronger than a long strip pillar as used in
highwall mining [Equation 3]. This does not make logical sense, and so raises a very significant question regarding pillar strength determination and assignment during design for industry to address. In hindsight, perhaps coal pillar strength research should have ceased with the Bieniawski equation from 1968 [Equation 1] based on the in situ testing of coal, rather than becoming an on-going statistical battle to develop new and supposedly “improved” pillar strength equations from failed cases databases, that in reality may provide no more than a slightly better answer to what may commonly be the wrong problem and at times, can be misleading.

While this did not occur in 1968 for obvious and understandable reasons, industry might be wise to focus more intently on other relevant pillar design, or more correctly, “overburden stability” design parameters, as outlined herein and in Reed, McTyer, and Frith 2016.

The final question Van Der Merwe 2006 poses relates to why a section of similarly sized coal pillars at Coalbrook that were directly adjoined to the large collapsed area did not similarly collapse (Figure 20). He offers that the only possible reason is that as the span across the pillars was reduced as compared to the main collapse area; the collapse did not propagate into this area. However, he also makes the following statement: “the fact that those pillars did not also fail cannot be explained by either the pillar safety factors or the presence of the dolerite sill based on current understanding. Clearly, a better understanding of the overburden stability is required. Had the overburden failed, then the pillars could not have survived, but current knowledge does not offer a method to evaluate the role of the overburden. For the time being, this question cannot be answered”.

![Figure 20: Details of the collapsed and non-collapsed areas at Coalbrook](Van Der Merwe 2006)

If the coal pillars in this un-collapsed area were of insufficient strength to prevent a collapse and the panel width was super-critical, even with the overburden containing a thick dolerite sill, and therefore was critically unstable to surface without the assistance of the coal pillars, only one credible possibility is seemingly left to explain why the area remained stable despite the absence of a barrier pillar between the collapsed area. Namely, the interaction between the coal pillars and the overburden maintained levels of horizontal stress within the overburden that provided a significant and critical contribution to overburden stability in addition to that of the coal pillars themselves, thereby preventing the overburden collapse.

In this regard, it is interesting to consider whether the alignment of the major horizontal stress may have coincided with the minimum span or width across the mined-out un-collapsed area. When reviewing the role of horizontal stress in South African coal mining, Frith 2002 found that the most predominant alignment of the major horizontal stress was closer to NS than EW based on observations and measurements at a number of underground mines.

Referring again to Figure 20, an alignment of the major horizontal stress east of north would closely align with the minimum span across the stable mined-out area. This combination logically represents the most stable possible overburden condition, as higher amounts of horizontal stress would need to be dissipated before the overburden could collapse via vertical shear at the extremities of the span.
Accepting the comments and judgements of Van Der Merwe 2006 as being essentially correct, this one small area that remained stable both during and after the Coalbrook collapse, conclusively proves that bord and pillar coal pillar stability problems cannot and should not be uniquely defined by full or even partial TAT representations, one exception to the rule potentially being sufficient to disprove the rule.

Decades later, it seems that the 1960 Coalbrook disaster may still provide us with valuable lessons if we are prepared to view the problem through a different lens.

**Implications to mine planning, layout design, and future research**

The final section of this paper considers, in general terms, the implications to future mine layout design as a result of these findings. Is there any benefit in undertaking research in order to overcome any of the limitations that are now seemingly inherent, to a varying extent, in our current bord and pillar type pillar design approaches?

The short-term answer to this question is probably "no". Major coal mining research institutions around the world that would have the resources to do the work are now largely absent from industry. It is also the case that the statistical approach to determining coal pillar strength equations has fortuitously, if perhaps unknowingly, compensated to some degree for some of the limitations by providing statistically-derived recommendations on what FoS or SF to apply in certain mining circumstances, irrespective of how accurate or inaccurate the coal pillar strength equations that evolved from the database may happen to be.

Nonetheless, should the nature of the pillar-loading scenario differ substantially from the majority of failed cases within a database (as per the Australian HWM and longwall chain pillar compression examples herein), the mine designer needs to be aware of this to ensure that the overall layout design contains suitable measures to compensate for any threats to mine or surface stability resulting from pillar strength equations over-estimating the true intact peak strength of the coal pillars. This frames the layout design problem as one of engineering adequate overburden rather than solely coal pillar stability and considers the coal pillars as one of several component in that problem.

Given that the paper has concluded that overburden reinforcement may be generally more appropriate for coal pillar design in non-caving mining scenarios such as bord and pillar and partial extraction, some historical context is required to explain why this may not have been realised much earlier:

1. In the aftermath of the Coalbrook tragedy, the need to develop credible pillar design methodologies to prevent a recurrence would have been enormous, both technically and politically. Spending many years undertaking research studies to generate all of the various geotechnical insights that the industry is incredibly fortunate to have today, would have been prohibitive and unimaginable. There was a need to provide a reliable solution relatively quickly. In that context, the researchers at that time and since should be recognised and congratulated for their achievements both now and into the future.
2. At the time of Coalbrook in 1960, the impact of horizontal stress in coal mine strata control was barely recognised, let alone proven and accepted. In fact, a Safety in Mines Research Advisory Committee research project came about in the early 2000’s as the role of horizontal stress in South African coal mine roadway roof control was still subject to industry debate at that time.
3. The insights in this paper have only come about as a result of nearly 60 years of international coal mine research and publications based on documented mining experiences—none of which were available in 1960.

The development of coal mine strata control knowledge and principles has been an on-going work-in-progress internationally for well over a century, but was undoubtedly accelerated after 1960 as a direct consequence of the Coalbrook disaster. Inevitably, as more mining experience-based research work is conducted and published, more engineers and scientists are exposed to real-world problems, rather than laboratory or computer-based simulations.
From time to time, significant realisations will inevitably emerge that challenge our views. This may be one such time. As a fraternity the question we need to consider is are we prepared to embrace a changing understanding of a problem and steer the ongoing search for improved knowledge in different directions as a direct consequence? Or conversely, do we prefer to stoically defend previous understandings, despite credible arguments that may render them incomplete, on the basis that it is perhaps more important to preserve the integrity of the past than look to the future with a new vision?

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ASSESSMENT OF COAL PILLAR STABILITY AT GREAT DEPTH

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ABSTRACT: The trial operation of the room and pillar method has been implemented at the shaft protective pillar of CSM coal mine, Czech Republic. Mining depth of the room and pillar trial ranged from 700 to 900 metres, being perhaps the deepest room and pillar coal mining in the world. An extensive monitoring system was implemented to measure the load profile across the coal pillars and the deformation characteristics in the pillars during mining. Stress-deformation monitoring was essential as this was the first application of the conventional room and pillar mining method within the Upper Silesian Coal Basin mines. The results of stress-deformation monitoring allowed pillar loading, yielding characteristics and coal pillar stability to be described. This data and other analyses are essential for establishing procedures for a safe room and pillar method of mining within the Upper Silesian Coal Basin. The results are also important for global mining, as many coal producers will reach higher mining depth in the near future.

INTRODUCTION

A considerable amount of coal reserves are located in protection pillars that lie under built-up areas within actively mined regions in the Czech part of the Upper Silesian Coal Basin (USCB). The commonly used controlled caving longwall mining method is not applicable in these areas because significant deformation of the surface is not permitted. For this reason the modified room and pillar method with stable coal pillars has been tested in order to minimise strata convergence. The trial operation of room and pillar method has been implemented in the shaft protective pillar (see Figure 1) where no mining was carried out in the past. Mining depth of the room and pillar trial ranged from 700 to 900 metres. It is perhaps the deepest room and pillar coal extraction in the world.

A stress-deformation monitoring program was essential as this was the first application of the conventional room and pillar mining method within the USCB. More than six kilometres of roadways were driven within three panels during the last three years. The last panel No. II was finished in September 2017. An extensive monitoring system was implemented to measure the load profile across the coal pillar and the deformation characteristics in the pillar during mining. Two monitored pillars diamond in shape and with slightly irregular sides were approximately 860 m² and 1200 m² in size in the first mined panel “V” and three monitored pillars of approximately 590 m², 590 m² and 730 m² in size were in the panel “II”. The monitored pillars were selected in different geotechnical conditions to study the behaviour of rock masses.

The adequate stability of coal pillars left behind is the prerequisite to minimise strata convergence and mining subsidence. The room and pillar mining method is usually implemented on the basis of gained experience and practices used elsewhere while taking into consideration different natural conditions and depths. The coal pillar sizes, calculated using accepted empirical methods (e.g. Salamon, 1970; Hustrulid, 1976; Bienawski, 1984; Mark and Chase, 1997 and Chase, Mark and Heasley, 2003), were uncertain due to complex strata geology. As there is no relevant experience of using this method in the USCB, an extensive monitoring system was implemented to enable the mining trial to continue safely.

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The geological setting in the area of shaft protective pillar CSM-North Mine is quite complex. The targeted coal seam (No. 30) is at a depth of approximately 700 m to 900 m below the surface. The thickness of the coal seam is extremely variable (from 180 to 520 cm) within the proposed mining area. The thickness of the coal seam ranges from 300 cm to 350 cm in the monitored pillars. The strata dip oriented in the North-East direction ranges from 8° to 17°. Occasionally the dip of the coal seam can reach up to 20°. There are several faults of regional importance in the area of the CSM-North shaft safety pillar (see Figure 1). The significant regional tectonic fault zone “Eastern Thrust” (Waclawik et al. 2013, Grygar and Waclawik 2011) divides the area of the protective pillar into two separate blocks with different geotechnical conditions. The overthrust zone has a significant impact on the deformation characteristic of coal pillars.

The vertical profile around coal seam No. 30 is shown in Figure 2. The immediate roof above the seam consists of a thin 0.1 m thick sandy claystone layer. This layer is relatively weak and disturbed with slickensides present on the surrounding bedding planes. Above this is a 5 m thick siltstone overlain with 6 m thick medium-grained sandstone and a 0.3 m thick coal
The immediate floor below the seam consists of a 0.5 m thick siltstone underlain by a 0.6 m thick coal seam No. 31. The interbedded siltstone and sandstone layers follow down to coal seam No. 32 located around 10 m below seam No. 30. The strata characteristics and geomechanical properties of rock mass have been predetermined for pillar loading and yielding characteristics. The strength of roof rocks is extremely high. Uniaxial Compressive Strength (UCS) of immediate roof (siltstone) determined from cores of exploratory boreholes ranges from 69 MPa to 129 MPa, 105 MPa on the average (Waclawik et al. 2017). UCS of overlying sandstone ranges from 70 MPa to 168 MPa, 116 MPa on the average. On the other hand the strength of floor rocks and the coal seam are lower. The strength of floor (clayey siltstone to siltstone) is variable depending on the content of clay. UCS of the floor ranges from 45 MPa to 80 MPa, 60 MPa on the average. UCS of coal ranges from 9.2 MPa to 22 MPa, 14 MPa on the average.

DESIGN OF GEOTECHNICAL MONITORING

The monitoring of the stress and deformation state in the rock mass is necessary precondition for a successful verification of the room and pillar method and its next application in conditions that may vary elsewhere in USCB. In the context of stress and deformation, the monitoring program covers the deformation of rock overlaying the room and pillar roadways, pre-mining stress measurements and stress change monitoring in rock and coal during mining, deformation of coal pillars, load on the installed cable bolts and roadway convergence monitoring. In addition, seismology and seismo-acoustic monitoring was carried out to characterise yielding of rock mass during and after mining. The precise monitoring of surface subsidence was also implemented within the shaft protective pillar.

Locality A

Monitoring was more complex in locality A due to missing previous experiences with strata behaviour during room and pillar excavation. The instrument locations are shown in Figure 2. To monitor roof deformation, fourteen pairs of 5-level multipoint extensometers monitored roof displacements (VE1 to VE14 in Figure 2.) and eleven strain gauged rockbolts (VS1 to VS11 in Figure 2) were installed at various locations. Four 5-level multipoint rib extensometers (VEH1 to VEH8 in Figure 2) were installed within each monitored pillar to measure displacements of all sides. Vertical and horizontal displacements together with the convergence measurements (VP1 to VP9 in Figure 2), changes in vertical pillar loads and periodic 3D laser scanning of the overall roadway displacements (roof, rib and floor heave) provided detailed data to evaluate panel stability.

To describe the pre-mining stress-state conditions of the coal pillar area, two 3-dimensional CCBO stress overcoring cells (Obara and Sugawara, 2003; Nakamura, 1999 and Stas, Knejzlik and Rambousky, 2004) were used (VCCBO1 and VCCBO2 in Figure 2) and eight 3-dimensional CCBM stress change monitoring cells (Stas, Knejzlik, Palla, Soucek and Waclawik, 2011; Stas, Soucek, and Knejzlik, 2007) were installed to measure stress changes during mining (VCCBM1 to VCCBM8 in Figure 2). Four 1-dimensional hydraulic stress monitoring cells were installed at various depths in each pillar to measure vertical stress (VSC1 to VSC8 in Figure 2), seven hydraulic dynamometer load cells measured the cable bolt loads installed at the roadway intersections (VD1 to VD7 in Figure 2).
Figure 2: Positions of the monitoring equipment in locality A and cross-section across monitored pillar V2 (on the right). Coal seams – blue, siltstones – green, sandstones – yellow (Waclawik et al. 2016).

Locality B

In locality B the number of monitoring instruments was reduced and supplemented with information from the monitored results in locality A. Due to minimal roof displacements during the whole time of monitoring in locality A, the strain gauged rock bolts and hydraulic dynamometers were not installed in locality B. The vertical 5-level multipoint extensometers were substituted with the cheaper 3-level multipoint extensometers. The instrument locations are shown in Figure 3. Other instruments used in locality B were the same as the instruments located in locality A. To monitor roof deformation, nineteen of 5-level multipoint extensometers (IIE1 to IIE19 in Figure 3) measured displacements within three monitored pillars (II1, II2 and II3). The convergence measurements in twelve stabilised profiles (IIP1 to IIP12 in Figure 3) were carried out. To monitor the stress-state of coal pillars three 3-dimensional CCBM stress change monitoring cells (IICCBM1 to IICCBM3) and five 1-dimensional hydraulic stress monitoring cells (IISC1 to IISC5 in Figure 3) were installed. Pre-mining stress-state was verified by two 3-dimensional CCBO stress overcoring cells.

RESULTS AND DISCUSSION

The displacements and development of deformation provide the data to assess coal pillar stability. The results of pillar displacement monitoring allowed the monitored pillars deformation characteristics to be defined. The data showed that the monitored coal pillar sides displaced substantially into the roadway mainly due to a large vertical stress field and the presence of weak slickensides above and below the seam. This mechanism caused large floor heave, rib convergence and relieved some of the confining stresses that usually build up within a pillar, therefore weakening the coal and causing the pillar to yield (Waclawik et al. 2016). The deformation mechanism presented here is supported by minimal displacement of roof rocks due to its high strength.
Figure 3: Instrument positions in the monitored pillars in locality B and cross-sections across monitored pillar II1, II2 and II3. Coal seams – grey, siltstones – blue, sandstones – yellow.

**Vertical displacement measurements**

The displacements of roof recorded by vertical extensometers reached insignificant values in both monitored areas (see Figures 4 - 6). The maximal value of displacement was only 7.6 mm in monitored locality A (maximum height of the top anchor was 8 m) for thirty two monitored months. The maximal values of displacement ranged between only 9 mm to 15 mm in monitored locality B (maximum height of the top anchor was 7 m) for 15 monitored months. These maximum displacements have been measured mainly above the intersections.

Figure 4: Roof displacements [mm] (on the left) and horizontal displacements at the right of coal pillars [mm] measured by 5-level extensometers in the monitored locality A.
Horizontal displacement measurements

The coal rib displacements recorded by horizontal extensometers are comparatively different within the monitored pillars (see Figures 4 - 7). In locality A, the largest displacements were recorded by horizontal extensometers installed in pillar V2. The displacement values ranged between 212 mm to 300 mm in monitored pillar V2. The coal rib displacements of the monitored pillar V1 ranged between 59 mm to 223 mm. These values indicate that the displacement of the coal pillar V2 is as large as monitored displacements of coal pillar V1, with higher area of loading. From the results recorded by the horizontal extensometers in location B, it is evident that the maximum horizontal displacement was 478 mm (II Eh5) in the monitored pillar II 2. Also, in the monitored pillar II 1, the relatively higher values were recorded (468 mm - II Eh1, 379 mm - II Eh3). Even in the monitored pillar II 3, which was formed last, the displacements values of around 300 mm (II Eh 9 - 344 mm, II Eh11 - 353 mm) were reached.

![Figure 5: Course of horizontal strata displacement in rib side measured by 5-level extensometers VEH2 (on the left) and II EH1 (on the right).](image)

In locality A, the major strata displacement zone occurred in the area 1.5–5 m from the pillar side (see Figure 5). The displacement at the depth of 0–1.5 m was much smaller due to the efficiency of rock bolts. In two cases (extensometers VEH7, VEH8) had no influence of the rock bolts and the maximum strata displacements occurred at the depth of 0–3 m into the pillar. The reason for this was considered to be the primary pillar damage by fractures in highly stressed ground.

In locality B, the major strata displacement zone occurred mainly at the depth of 5-8.5 m from the side (see Figure 5). The significant strata separation was recorded at the deepest monitored zone 8.5-12 m (7.5-10 m in monitored pillar II2), which indicated that the monitored pillars were totally fractured. In most cases there was no measured rockbolt influence on coal behaviour and the significant strata separation occurred at the depth of 0–1.5 m into the pillar.
Figure 6: Roof displacement [mm] (above) and horizontal displacement of coal pillars [mm] measured by 5-level extensometers in the monitored locality B.

Figure 7: Rib movement [mm] from the convergence station in the monitored locality A (above) and locality B.
Measurements of roadway deformation

The convergence stations were placed in the centre of the monitored side coal pillars (see Figure 2 and 3). The three convergence stations (VP1, IIP3 and IIP5) were installed at the roadway intersections. For the purpose of convergence measurements 2.4 m long steel rock bolts were installed in the roadway roof and rib. The measured convergence values were different both within all convergence stations and at particular convergence station. The maximum horizontal deformation of 548 mm (convergence station VP8) was recorded in locality A during thirty two monitored months. In comparison, the maximum horizontal deformation of 838 mm (convergence station IIP1) was recorded in locality B during only fifteen monitored months. Dynamics of deformation changes were more rapid in locality B due to the smaller size of the monitored pillars and added row of pillars (two rows at locality A and four rows at locality B). The presence of thrust zone in roof of monitored pillar (see Figure 3) probably also influenced the deformation characteristic. The influence of thrust zone on stress-deformation state will be analysed in the near future using numerical models.

The compact pulsed terrestrial laser scanner Leica Scan Station C10 was used for monitoring the of time dependent changes of coal pillars (Kukutsch et al. 2016). 3D laser scanning has been carried out within 11 stand-alone measurement campaigns in the case of locality A and within five campaigns in the locality B. From the beginning, the time gap between campaigns ranged from five to six weeks, followed by a widening of the campaign interval to three, later four months.

![Figure 8: Example of laser scanning of the roadway profile - roadway V300501, locality A, (on the left) and roadway II3005, locality B (on the right)](image)

In each campaign scanning was performed from at least six consecutive scan positions with scanning resolution at 10 mm / 10 m. As a result more than 14.5 million spatial points for each of the scan positions were detected and almost 90-120 million spatial points in one scanning campaign. Repeated measurements allow detecting, with a millimetre accuracy, of changes in pillar size, roadways profile and in particular the possibility of graphically expressing the dynamics of the pillar - its displacement.

The laser scan results from several subsequent surveys indicated significant coal rib convergence and floor heave. In some places the lateral displacements measured mostly at the lower rib exceeded more than 80 cm. As expected, the roof displacements were not registered. The data provided by laser scanner allowed the character of roadway deformation to be described. The results compared well with the rib extensometers and convergence measurements. The 3D laser scanning technology enables very detailed analysis to be performed to evaluate the long term coal pillar stability (e.g. Kajzar et al. 2017).
Dynamics of displacement changes

In addition to absolute values of pillar displacements, data from horizontal extensometers, convergence measurements and the 3D laser scanning provided important information on dynamics of coal pillar behaviour. Concerning the dynamics, we could see a decrease of displacements during the whole evaluation period.

Figure 10: Rate of the V2 coal pillar displacements versus time.

In the locality A, the monthly gain of displacements reduced up to 25 times during monitoring period of 32 months (see Figure 10). The monthly gain of displacements stabilised at 3 mm/month during the last eighteen month of the monitoring period. Similarly, the 3D laser scanner provided convergence measurements and showed the dynamics of roadway deformation. Continuous deformation processes indicate, that yielding of coal pillars is still in progress therefore the long-term stability of coal pillars has not been established yet. The monitoring of conditions at locality A still continues.
In the locality B, the monthly gain of measured displacements reduced up to 25 times during the first nine months (see Figure 11). The monthly gains of displacements stabilised at 2 mm/month. Finally, the monthly gains of displacements have been significantly influenced by roadway advance. Due to significant deformation of the main roadways it was decided to form a fourth row of coal pillars at the last phase of monitoring. The new roadway II3006 was driven and the resulting stress re-distribution reactivated deformation of monitored coal pillars. The monthly gain of displacement increased up to 62 mm per month in the most influenced monitored pillar II3. However, the follow-up monitoring was not possible due to closure of the panel II.

**Stability of coal pillars**

Based on the development of deformation characteristics, the stability of the left coal pillars can be concluded. Due to the long-term monitoring (up to 32 months) and the development of deformations, it is possible to determine the *operating stability* and *long-term stability* of coal pillars. It can be stated that the operating stability of not only the monitored coal pillars but also of all coal pillars left in the trial area of the room and pillar method has been confirmed. Any of the coal pillars have not been destroyed during the whole monitoring period.

Due to the fact that the deformation process is still currently in progress and that the size of the monthly gains of deformation has not significantly diminished during the last eighteen monitored months, the long-term stability of coal pillars have not been confirmed yet. For this reason the monitoring continues at the locality A, as the panel is still accessible.

**CONCLUSIONS**

The room and pillar method was trialled in the shaft protective pillar at the CSM Mine located in the USCB. Coal pillar monitoring was essential as this was the first application of the room and pillar mining method in USCB mines at great depth. In total, five coal pillars located in seam No. 30 were intensively monitored to ensure stability of the panel and safe mining procedures. Based on the measurements, numerical modelling and other analyses were possible to assess stability of the coal pillars at the great depth. The results are also important for global mining, for the largest coal producers will reach higher mining depth in the near future.

Based on the long-term monitoring (up to 32 months), it was possible to determine the operating stability and long-term stability of coal pillars. Safe operating stability has been
proven for all coal pillars formed within the trial operation of the room and pillar method. The long-term stability of coal pillars has not been confirmed yet because a small deformation process in coal pillars is still in progress. Further ongoing monitoring of pillar movement at locality A is in place as the locality is still accessible and monitoring devices are still operating.

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REFERENCES

MECHANICAL PROPERTIES OF COAL MEASURE ROCKS CONTAINING FLUIDS AT PRESSURE

Ian Gray¹, Xiaoli Zhao², Lucy Liu³

ABSTRACT: The sedimentary rock that comprises coal measures has quite nonlinear, elastic, stress – strain characteristics. It is also affected by the fluid pressure within it. The fluids act in two quite separate ways. The first way in which fluid acts is in a poroelastic manner while the second is within fractures. These effects are important in rock behaviour, extending from the deformation around a roadway to failure within an outburst. This paper presents the results of detailed laboratory studies into coal and sedimentary rock properties. It relates these to the real situations seen in mining.

INTRODUCTION

All sedimentary rocks exhibit variability in both Young's moduli and Poisson's ratios under different stresses. Some are also quite anisotropic. This nonlinear, anisotropic, elastic behaviour is extremely important in determining the stresses within the rock mass both in the virgin state and as a response to mining. Fluid pressure within the rock mass is also important as it is a component of effective stress.

Determining the rock properties is quite complex. The options are uniaxial testing, triaxial testing and hydrostatic testing. The usual procedure is to rely on simple uniaxial testing. This however tends to give quite inadequate results. Uniaxial testing only enables the axial modulus to be determined over a very limited stress range before the sample starts to fail. This failure is accompanied by a rapid increase in Poisson's ratio. The single measurable value of Poisson's ratio cannot therefore be gauged accurately. It is also impossible to subject a uniaxial sample to the effects of fluid pressure.

Triaxial testing permits the Young's modulus and Poisson's ratios to be determined by axial and radial (confining stress) loading of a strain gauged core sample in a triaxial cell. The core is loaded sequentially with changes in axial and then radial pressure to enable the determination of the Young's moduli and Poisson's ratios. Mathematics has been developed to determine these values on the basis that the rock behaves as an orthotropic material. This form of triaxial testing is not the same as that used to determine the ultimate strength parameters of the rock.

Hydrostatic testing is suitable to measure the behaviour of rock fragments. This is especially common in coal. This method involves strain gauging a fragment with multiple rosettes, casting in a soft resin, and then hydrostatically loading it while recording the strain behaviours.

Figure 1 shows examples of these three test methods. The left photograph in Figure 1 shows all of the problems of dealing with a piece of weak disintegrating coal core in uniaxial testing. The sample is too short, it is dimensionally uneven and the ends cannot be cut parallel and have to be built up with plaster. The test is slightly better than useless for determining material properties. The middle photo shows a similar coal sample that has been fitted with strain gauges and is contained in silicone resin prior to hydrostatic testing. The right photo shows a good core of a stronger coal that is ready to triaxially tested.

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STRESS STRAIN RELATIONSHIPS

A solid may be subject to six stresses, which produce six strains. The relationship between stress and strain therefore contains 36 components. Determining all of these is practically impossible. However if the assumption is made that the rock is orthotropic this can be simplified to twelve unknowns. If one of the axes of symmetry can be identified, such as that perpendicular to a bedding plane, this is reduced to nine as shown in Equation (1), which relates principal stresses and strains. The symmetry of this matrix means that the relationship of Equation (2) applies.

\[
\begin{bmatrix}
\Delta \varepsilon_{11} \\
\Delta \varepsilon_{22} \\
\Delta \varepsilon_{33}
\end{bmatrix} =
\begin{bmatrix}
\frac{1}{E_1} & -\frac{v_{21}}{E_2} & -\frac{v_{31}}{E_3} \\
-\frac{v_{12}}{E_1} & \frac{1}{E_2} & -\frac{v_{32}}{E_3} \\
-\frac{v_{13}}{E_1} & -\frac{v_{23}}{E_2} & \frac{1}{E_3}
\end{bmatrix}
\begin{bmatrix}
\Delta \sigma_{11} \\
\Delta \sigma_{22} \\
\Delta \sigma_{33}
\end{bmatrix}
\]

(1)

\[
\frac{v_{ij}}{E_i} = \frac{v_{ji}}{E_j}
\]

(2)

The term \( v_{ij} \) refers to the Poisson’s ratio associated with dilation in the \( j \) direction brought about by loading in the \( i \) direction.

These two relationships mean that there are six unknowns to be solved. Given two potential loading cases – axial and radial, and three radial strains, this set of equations cannot be solved for Young’s moduli and Poisson’s ratios. Making the assumption of Equation 3 that introduces the concept of a geometric mean value of Poisson’s ratio makes it possible to solve all values of Young’s modulus and Poisson’s ratio.

\[
v_a^2 = v_{ij}v_{ji} = v_{jk}v_{kj} = v_{ik}v_{ki}
\]

(3)

By manipulation of these equations the solution for Young’s modulus can be described by Equation 4 which can be fully solved in the case of axial and radial loading steps. This is achieved by a nonlinear solution process in which the value of \( v_a \) is adjusted to provide a best fit between the axial modulus derived from radial loading and that derived from axial loading.

\[
E_i = \frac{1}{\Delta \varepsilon_i} \left( \Delta \sigma_i - \sqrt{\frac{E_i}{E_j}} v_a \Delta \sigma_j - \sqrt{\frac{E_i}{E_k}} v_a \Delta \sigma_k \right)
\]

(4)
In hydrostatic testing no solution to the value of \( v_a \) can be obtained from the test process and \( v_a \) has to be estimated. Using this estimated value of \( v_a \) the values of Young’s moduli and Poisson’s ratios can be derived using Equation 5.

\[
E_i = \frac{\Delta \sigma_i}{\Delta \varepsilon_i} \left( 1 - \sqrt{\frac{E_i}{E_j} v_a} - \sqrt{\frac{E_i}{E_k} v_a} \right)
\]

(5)

**Effect of fluid pressure**

Fluid pressure operates on open spaces within the rock mass. A fluid pressure change may lead to a deformation of the rock mass, which then behaves as though it were a change in stress. It may also act directly on open spaces or fractures changing the normal stress within these. Either of these effects may be described by Equation 5 (Gray et al, 2017).

\[
\sigma'_{ij} = \sigma_{ij} - \delta_{ij} \alpha_i P
\]

(6)

Where:

- \( \sigma'_{ij} \) is the effective stress on a plane perpendicular to the vector \( i \) in the direction \( j \).
- \( \sigma_{ij} \) is the total stress on a plane perpendicular to the vector \( i \) in the direction \( j \).
- \( \delta_{ij} \) is the Kronecker delta. If \( i \neq j \) then \( \delta_{ij} = 0 \), while if \( i = j \) then \( \delta_{ij} = 1 \).
- \( \alpha_i \) is a poroelastic coefficient affecting the plane perpendicular to the vector \( i \). It’s value lies between 0 and 1.
- \( P \) is the fluid pressure in pores and fractures within the rock.

In the poroelastic case Equation 1 can be re-written using Equation 6 as Equation 7.

\[
\begin{bmatrix}
\Delta \varepsilon_{11} \\
\Delta \varepsilon_{22} \\
\Delta \varepsilon_{33}
\end{bmatrix} =
\begin{bmatrix}
\frac{1}{E_1} & -\frac{\nu_{12}}{E_2} & -\frac{\nu_{13}}{E_3} \\
-\frac{\nu_{12}}{E_1} & \frac{1}{E_2} & -\frac{\nu_{23}}{E_3} \\
-\frac{\nu_{13}}{E_1} & -\frac{\nu_{23}}{E_2} & \frac{1}{E_3}
\end{bmatrix} \begin{bmatrix}
\Delta \sigma_{11} \\
\Delta \sigma_{22} \\
\Delta \sigma_{33} \\
\end{bmatrix} - \Delta P \begin{bmatrix}
\frac{1}{E_1} & -\frac{\nu_{12}}{E_2} & -\frac{\nu_{13}}{E_3} \\
-\frac{\nu_{12}}{E_1} & \frac{1}{E_2} & -\frac{\nu_{23}}{E_3} \\
-\frac{\nu_{13}}{E_1} & -\frac{\nu_{23}}{E_2} & \frac{1}{E_3}
\end{bmatrix} \begin{bmatrix}
\alpha_1 \\
\alpha_2 \\
\alpha_3
\end{bmatrix}
\]

(7)

Equation 7 describes the deformation of rock due to changes in stress and fluid pressure. The poroelastic coefficients \( \alpha_i \) lie between zero and unity.

It is possible to solve Equation 7 in the triaxial testing by axial and radial loading stages and by introducing fluid injection. \( E_1, \nu_{12} \) and \( \nu_{13} \) are determined from axial loading, and \( E_2, E_3, \nu_{23, 23, 23, 23, 23} \) are deduced through the radial loading step. The strain changes associated with fluid injection may be measured and provide sufficient information to derive a solution for the poroelastic coefficients.

If a rock mass contains an open joint then the effect of fluid pressure acting within it may be better described by Equation 6 alone where \( \alpha_i \) may be thought of as the ratio of open fracture to total area. In reality the rock mass will never contain a totally open joint and some poroelastic effect will be present; the values of \( \alpha_i \) may be expected to vary as the joint opens and closes.

It should be appreciated that in typical sedimentary rock with nonlinear characteristics the values of all Young’s moduli, Poisson’s ratios and the poroelastic coefficients vary with the state of stress.

**EXPERIMENTAL PROCEDURES**

Uniaxial testing may follow a standard such as AS 4133.4.3.1 (Australian Standard, 2009). However, proper attention to the testing process and analysis process is required. The stress and strain values obtained from a uniaxial cyclic loading test are shown in Figure 2. The sample is a typical sandstone and the results of the test are extremely non-linear. This non-linearity is generally ignored and the results are quoted in terms of a tangent and secant.
modulus at 50% of ultimate strength. This ignores the fact that the tangent modulus frequently increases four fold before it begins to decline with the onset of failure. This onset of failure is frequently associated with the value of tangent modulus exceeding 0.5.

Figure 2: Uniaxial test – stress versus strain plot

Triaxial testing is conducted on core sample fitted with three rosettes disposed at 120 degrees around the circumference. Typically four loading tests are conducted. The test sequence is first to load in relatively equal stages of axial and radial (confining) stress to avoid causing the sample to fail. The sample is then unloaded in a similar manner. A second stage involves loading but with a lower confining stress – typically 80% of axial. This procedure is then repeated with radial stress at 60%, 40% and 25% of axial before testing with axial stress alone. This procedure minimises the risk of the sample failing early in the process through excessive shear stress. In addition to the axial and radial loading cycles, fluid, usually nitrogen or helium, is injected into the rock between direct loading cycles to enable the determination of poroelastic behaviour.

Figure 3 shows the loading cycle where the radial stress is 80% of axial stress. This test also includes the injection of nitrogen into the sample.

The hydrostatic test involves fitting strain gauges to the sample. These may be rosettes placed on orthogonal faces or a rosette on a bedding plane and single gauges perpendicular to the bedding plane. The fragment is then set in silicone resin, hydrostatically loaded, and the strain monitored as shown in Figure 4. This is usually conducted in a cyclic loading process so that the loading and unloading Young’s moduli may be determined.
Figure 3: Triaxial test to determine Young’s moduli, Poisson’s ratios and the poroelastic coefficient. X axis time (s), Y axis stress (AP and CP in kPa), and microstrain.

Figure 4: Hydrostatic test. X axis time (s), Y axis pressure (kPa) and microstrain.

EXPERIMENTAL RESULTS

A coal sample from the Goonyella Middle (GM) seam in the Bowen basin has been tested both uniaxially and hydrostatically. Coal samples from the Bulli seam in the Sydney Basin and a porous Hawkesbury sandstone sample from Sydney have been tested triaxially.

Uniaxial test results

The results from non-cyclic uniaxial testing a sample from the GM seam are shown in Figure 5. The GM seam sample was of a similar poor form as that shown in the left hand photo of Figure 1. The sample’s stiffness changes during the test with a general tendency to increase.
but with obvious stages of failure. The right side of Figure 5 shows that Poisson’s ratio suddenly increases as failure approaches.

The sample is clearly nonlinear in its behaviour and yet conventional reporting would typically provide a single value based on a fixed percentage (usually 50%) of the ultimate stress. Figure 5 indicates that the secant Young’s moduli and Poisson’s ratios vary more than two fold over the range of the test. The tangent Young’s moduli fluctuate greatly and vary more than four fold over the test. The tangent Poisson’s ratio increases significantly with stress. When stress approaches 8 MPa, the value of Poisson’s ratio reaches 0.5, meaning that the sample is behaving plastically at this stress level. At stresses above this level the sample is dilating. This behaviour is quite normal for coal under uniaxial testing. Better samples of sandstone or siltstone will give smoother change in tangent modulus and a gradually increasing Poisson’s ratio.

![Figure 5: Results from uniaxial test – Young’s modulus (left) and Poisson’s ratio (right)](image)

**HYDROSTATIC TEST RESULTS**

Figure 6 shows the result of a hydrostatic test of the GM seam coal sample. The secant Poisson’s ratio for the uniaxial test ranges between 0.1 and 0.3 and a value of a geometric mean Poisson’s ratio $v_a$ of 0.2 is used in the analysis of the hydrostatic testing. The sample is obviously nonlinear and anisotropic. The axial Young’s modulus is the lowest among all the values, being 500-1000 MPa lower than the minor transverse Young’s modulus. It is 200 MPa less than that of the major. The sample is nonlinearly elastic with the values of three Young’s moduli varying two to more than three fold over the range of the test.

![Figure 6: Hydrostatic results – Young’s modulus (left) and Poisson’s ratio (right) plotted with respect to hydrostatic stress. The assumption used in analysis is that $v_a=0.2$.](image)
Triaxial elastic test results

Figure 6 shows the axial Young’s modulus of a dull Bulli coal sample plotted against axial and confining stress. Figure 7 shows the major transverse Young’s modulus, which is similar in value to the axial modulus. Both moduli show a large increase in value with stress. The axial modulus is dependent on both axial and confining with axial stress having a slightly more dominant effect. The major transverse modulus appears to be slightly more dependent on confining stress.

![Figure 6: Axial Young’s modulus (E₁) of coal plotted as an isopach with respect to axial and confining stress.](image1)

![Figure 7: Major transverse Young’s modulus of coal(left) and porous sandstone (right) plotted as an isopach on axial and confining stress.](image2)

The second sample is a porous medium grained Hawkesbury sandstone which has been cored approximately perpendicular to the bedding plane. The left picture in Figure 8 shows the axial Young’s modulus (E₁), which is perpendicular to the bedding plane, plotted with...
respect to axial and confining stress. $E_1$ increases with stress and is primarily a function of the axial stress. The major transverse Young’s modulus shown in the right picture in Figure 8 stiffens with stress but is primarily a function of the confining stress. In this sample therefore the value of Young’s modulus appears to be controlled by stress in the direction of the modulus being determined.

Not all sandstone samples behave in this way. Some have shown values of Young’s moduli that are a function of axial and confining stress.

Figure 9 shows the trend of geometric mean Poisson’s ratio ($\nu_a$) for the coal sample (left) and the porous sandstone (right). The values of $\nu_a$ in the coal vary little, but are quite dependent on the shear stress in the porous sandstone. The latter trend is more frequently observed.

Figure 10 shows the values of Young’s modulus determined perpendicular to the bedding plane for a wide number of coals plotted against mean stress. The general trend is for modulus to increase with stress typically four fold, but in some cases ten fold. The more dramatic changes in stiffness are associated with softer coals which tend to be weaker and contain more structure.
Poroelastic test results

The poroelastic coefficient describes how the rock or coal deforms with internal fluid pressure as described in Equation 7. The effective stress may be derived from Equation 6. This value of effective stress does not describe the stress at a granular level.

Figure 11 shows the axial poroelastic coefficient of coal and sandstone in the direction of the axis of the samples. The value of the poroelastic coefficient in coal is much lower than that in the porous sandstone. Both values tend to increase with shear stress and decrease with confining stress though to quite different extents.

Extensive tests have shown that the poroelastic coefficient usually lies between 0 to 0.9 in rock, and 0.1 to 0.3 in coal. In coal it tends to be more dependent on the state of stress than in rock.

CONCLUSIONS

This paper compares the mathematics and experimental procedures to obtain orthotropic elastic parameters including poroelastic behaviour using uniaxial, triaxial and hydrostatic testing of coal and sandstone.
Results from the three types of tests show that the mechanical properties of rock vary with stress and may be anisotropic. The effects of fluid pressure within the rock may also be important. Virtually all of Young’s moduli increase with stress. This variation may be up to ten fold in weaker coals but is frequently four fold. Anisotropy has not been found to be great, usually being less than 1.5:1.

This work highlights the inadequacy of the linear elastic models, which ignore fluid pressure and are the basis for the majority of rock mechanic designs at the moment. These simplifications for numerical convenience are significantly in error.

The stiffness of some coals is extremely important in the way in which it may store strain energy. Under the same strain conditions stiffer coals will develop far more stress and higher strain energies. This has important consequences for coalbursting.

The increase in poroelastic coefficient with shear is of importance too as it provides a means by which fluid pressures may act within the coal or rock mass leading to failure. This is as important to outbursting as it is to slope stability.

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COAL AND ROCK BURSTS – SIMILARITIES AND DIFFERENCES WHEN CONSIDERING THE SUDDEN COLLAPSE OF THE SIDES OF EXCAVATIONS

Ross Seedsman

ABSTRACT: Using metalliferous terminology sudden collapses of the sides of coal mine roadways are likely to be strain bursts, plus the possibility of some kinematic failures or slumps. Brittle failure in coal induces vertical slabs parallel to the excavation boundary, which if unsupported, can topple or slide into the roadway. In Australian coalmines, the potential collapse of the ribs during strain bursting is controlled by the routine rib support that is installed off the continuous miners. Mine seismicity may be the trigger for additional brittle failure. The energy released by brittle failure can be absorbed by a bolted and meshed rib or may cause ejection. The association of the term “coal burst” with high velocity ejection of coal may be preventing the identification of sudden rib failures, which are the simple collapse under gravity of kinematically acceptable wedges that have dimensions greater than the length of the installed bolts. Vertical pillar deformations later in the mining cycle may generate additional brittle failure or load existing kinematically acceptable slabs or wedges causing them to collapse.

INTRODUCTION

Differing from typical metalliferous excavations, coalmine roadways are rectangular in shape and are formed in a transversely isotropic rock mass excavated using continuous miners. In Australian underground coal mines rib and roof bolting is conducted about 3 m from the active mining face behind the miner cutter head and coal gathering system; the mining method places the development workforce within 1.3 m of the ribs which are typically supported with mesh and short bolts. Subsequently, during either longwall mining or pillar extraction, the workforce can be within 2 m or 3 m of an active coalface that is not supported. Geological faults and other structures may need to be traversed to access “undisturbed” areas for coal extraction. By way of contrast, a typical metalliferous roadway is horseshoe shaped and excavated with drill and blast in rock masses that are assumed to be isotropic. The typical ground support consists of bolts and fibrecrete with mesh introduced when there are concerns with rock bursts. Mineralisation is often associated with large scale faulting.

The burst terminology is different between the two mining sectors. According to Canbulat, et al. (2016) a pressure (or coal) bump is a form of dynamic release of energy within the rock (or coal) mass in a coal mine due to either intact rock failure or failure/displacement along a geological structure that generates an audible signal, ground vibration, and potential for displacement of existing loose or fractured material into mine openings. A pressure (or coal) burst is a pressure bump that actually causes consequent dynamic rock/coal failure in the vicinity of a mine opening, resulting in high velocity ejection of this broken/failed material into the mine opening. In metal terminology (Kaiser 2016) strain bursts are the result of a sudden bulking process associated with rock failure that may be triggered by a seismic event (itself possibly a fault slip) but are primarily the result of the tangential stress near an excavation exceeding the capacity of the unconfined or lightly confined rock mass due to excavation advance or nearby mining (the latter called a pillar burst). In the absence of support there may be ejection of the failed rock. Shakedown is the subsequent collapse of failed rock in response to a seismic event.

So in summary, a coal bump is a felt seismic event and a coal burst involves not only failure of the coal but also requires an unspecified high velocity ejection. In contrast a rock strain burst is defined as a failure process that may or may not be caused by a seismic event and the consequences of which may or may not be ejection depending on the installed support.

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University of Wollongong, February 2018
These may appear to be subtle differences but they become important when designing possible control mechanisms.

Burst Hazard

A simple empirically-derived relationship used in rock burst assessments (Kaiser et al, 1996) is the ratio of magnitude of the maximum stress at the excavation boundary relative to the laboratory uniaxial compressive strength (UCS) - referred to as SL<sub>UCS</sub> with a value of 0.3 indicating the onset of a bursting hazard and 0.7 being the limit of the empirical data. The Q system (Barton et al, 1974) identifies a rockburst hazard by the reciprocal of this ratio with a mild burst at a SL<sub>UCS</sub> of 0.2 and a heavy burst at 0.4; these were later changed to 0.33 and 0.5 (Grimstead and Barton, 1993). The Coal Strength Index (CSI, Seedsman, 2012) is the ratio of the coal UCS to the vertical stress and conversion to SL<sub>UCS</sub> requires consideration of the shape of the opening, the stress field in coal, and transverse isotropy and may be approximated as equal to 5/CSI for the ribs. In Figure 1, it can be seen that a moderate burst hazard according to the definitions in hard rock is reached in coal mines at about 100 m depth and exceeds the metaliferous empirical relationship by 150 m depth. In the metaliferous sphere, rockbursts are subdivided into self-initiated strain bursts, seismically triggered strain bursts, and dynamically loaded strain bursts. As mentioned earlier these definitions are independent of possible ejection after failure. For coal, Iannacchione and Zelenk (1995) refer to three mechanisms – loss of confinement, seismic stress, and excessive pressure.

Seismicity

All failure in rock/coal masses can generate a seismic event of some magnitude. A self-initiated strain burst produces a seismic event just as mining-induced movements along a distant fault structure can produce a more energetic event such that when it arrives at an excavation boundary it initiates a strain burst.

Up until recently, support design practice in metal mines was based on estimating ejection velocities using case studies of presumed ejection velocities and the distance to seismic events. Ejection is considered possible if the Peak Particle Velocity (ppv) exceeds 1 m/s and it is common practice to assume a default design value of 3 m/s. For rock ejection the method advocated in Kaiser et al (1996) would require a Richter 2 event at 15 m from an excavation. In a major change in the design approach, Kaiser (2017) proposes that ejection velocities are simply related to the rapid onset of brittle failure. For example if 0.5 m thickness of rock undergoes rapid brittle failure in 0.1 seconds and then bulks by 20 % the resulting ejection velocity would be 1 m/s. In this model the mine seismicity may trigger the brittle failure but does not by itself accelerate the rock.

Microseismicity studies in Australian coalmines do not quote Richter magnitudes. The GeoScience Australia earthquake database includes Richter 2 and 3 events in the Appin and Cessnock areas; unless the events were very close to an excavation the likely impact according to rock burst knowledge would have been bulking but not ejection. Some
appreciation of what can be felt in underground coal mine roadways can be gleaned from Table 1 which is derived from the Mercalli scale for earthquakes. Based on this scale the author has been exposed to events up to “Strong” (possibly accelerations of up to 0.18 g).

Table 1: Possible felt scale for coalmine bumps

<table>
<thead>
<tr>
<th>Qualitative Description (ACG 2008)</th>
<th>Mercalli perceived shaking and seismic coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground shaking felt close to the event. Felt as good thumps or rumbles. May be felt remotely from the source event (more than 100 metres away).</td>
<td>Light (IV) 0.014-0.039g</td>
</tr>
<tr>
<td>Often felt by many workers throughout the mine. Should be detectable by a seismic monitoring system.</td>
<td>Moderate (V) 0.039-0.092g</td>
</tr>
<tr>
<td>Vibration felt and heard throughout the mine. Bump may be felt on surface (hundreds of metres away), but may audible on surface. Vibrations felt on surface similar to those generated by a development round.</td>
<td>Strong (VI) 0.092-0.18g</td>
</tr>
<tr>
<td>Felt and heard very clearly on surface. Vibrations felt on surface similar to a large production blast. Events may be detected by regional seismological sensors located hundreds of kilometres away.</td>
<td>Very strong (VII) 0.18-0.34g</td>
</tr>
<tr>
<td>Vibration felt on surface is greater than large production blasts. National seismic stations can usually detect events of this size.</td>
<td>Severe (VIII) 0.34-0.65g</td>
</tr>
</tbody>
</table>

DEFORMATION AND FAILURE OF COAL RIBS

Rock burst literature proposes that the ejected rock has undergone brittle failure at the excavation boundary with the interpretation that seismic loading by itself does not increase the volume of failed material. Rock bursts are associated with brittle failure in hard rock and this section discusses the brittle failure in the ribs of coal mine roadways.

Observations

At shallow and moderate depths the sides of coal mine roadways are characterised by closely-spaced mining-induced fractures that strike parallel to the roadway direction (Figure 2); at greater depths the ribs may be so broken that the slabs are not readily discernible. The thickness of the slabs is typically less than about 10 cm. The mining-induced fractures exploit persistent joints in the coal when the roadway is driven sub-parallel. At other orientations, the mining-induced fractures are quite distinctive with a rough/stepped profile compared to smooth/planar profile for natural joints. The fact that the slabs have a vertical continuity indicates that intrinsic cleating of the coal is not a discontinuity in a geotechnical sense. In fact, the RQD of coal is typically 100 even though coal itself is typically strongly banded with alternating bright and dull layers.

Extensometry

A typical extensometer plot for a coal rib in a development roadway at 480 m depth (Figure 3a) indicates that most of the movements are within 1 m of the excavation boundary. It is noteworthy that as the longwall retreated past the instrumentation the depth of movement increased from 0.5 m to 1.0 m and the horizontal movement at the boundary increased from 15 mm to 80 mm. If the non-elastic movements are assumed to be those in excess of 10 mm and recognising the resolution of the extensometer, Figure 3b suggests that the maximum depth of failure apparently does not increase with depth of cover between 140 m and 480 m depth.
Examples of coal ribs.

Figure 2: Examples of coal ribs.

(a) Thin slabs formed in a bolted and strapped coal seam rib at about 400 m depth of cover
(b) Slabs formed in the sides of an unbolted coal rib at about 65 m depth

Figure 3: Deformations in coal ribs

Elastic deformations

Figure 3c and 3d presents the result of a finite-element analysis of a rectangular roadway in a transversely isotropic homogeneous continuum with a horizontal to vertical stress ratio of 0.5 for a number of values of the ratio of Young’s modulus to the Independent Shear Modulus (E/G) and normalised to a Young’s modulus of 1 GPa. The elastic deformations increase with increasing values of the E/G ratio. In work on coalmine roofs, an E/G value of 30 has been used for moderately to thickly bedded units and 100 for thinly bedded units. For an extensometer with a 6 m length, the resolved elastic movements at the excavation boundary would be in the order of 2.5 mm for every 100 m depth of cover assuming a typical coal modulus of 2 GPa. Applying these results to the extensometer data in Figure 3a, the limit of elastic movement is 12 mm and hence non-elastic movements (failure) are located within about 0.5 m of the excavation boundary.
Brittle failure

Strain bursts in rock are caused by the sudden creation of a zone of stress-fractured rock (Kaiser, 2016) and this stress fracturing can be modelled using brittle rock concepts in what is referred to as the inner shell of an excavation. The UCS of coal varies from about 5 MPa for some high quality coking coals to about 40 MPa for dull thermal coals (Young’s modulus/UCS ratio is typically in the range of 100 to 200). The possible independence of the depth of failure with respect to depth of cover (Figure 3b) suggests that a suitable failure criterion would invoke a stress ratio and not solely a deviatoric stress. The spalling limit component of the S-shaped failure criterion (Kaiser and Kim, 2008) is such a criterion. Laboratory testing of an Australian thermal coal has revealed evidence of a spalling limit value of 38 (Buzzi et al, 2014).

Simple boundary element analyses using the Transverse Isotropic Brittle failure (TIB) criterion (Seedsman, 2017) with an E/G ratio of 30 and a spalling limit of 38 produce a failure zone that appears somewhat similar to that developed in coal mine roadways – greater stress failure at top and bottom corners, and a vertical boundary at mid-height (compare Figure 4a with Figure 2). A spalling limit of 38 gives a depth of failure similar to that recorded in extensometry.

![Image](image_url)

(a) Contours of the strength factor as defined by Examine2D with respect to damage initiation/cohesion assumption and limited by a spalling limit of 38 (CSI=5.4, E/G=30)

(b) Depth of rib failure as a function of CSI and SL_{UCS}

(c) The gabion concept for ground control in strain burst ground (Kaiser, 2016)

(d) Impact of transverse isotropy and stress ratio on the depth of brittle failure in coal ribs

Figure 4: TIB analysis of coal ribs

Similar to the rock strength index (Seedsman, 2014), the depth of failure in coal ribs can be related to the CSI. Figure 4b shows that the depth of failure does not increase for CSI values less than about 5.5 as this is when brittle failure is determined by the spalling limit independent of the cohesive strength. Using the TIB failure criterion, the maximum depth of
failure depends on the horizontal to vertical stress ratio (K) and also on the value of the E/G ratio (Figure 4d). The increase in the depth of failure at very low K values is of significance when considering the stress conditions under pillars in multiple seam mining (Seedsman, 2017).

These analyses are 2-dimensional plane-strain and represent the conditions likely to develop at about 10 m - 15 m from the mining face. Three-dimensional modelling of rectangular roadways in an isotropic continuum suggest that stresses at the time the rib bolts are installed in Australian mines will be about 70% of the plane strain values: hence the need for rib support to be installed off the continuous miners to address the possible formation and immediate collapse of the slabs induced by brittle failure.

SUMMARY

The concepts of brittle failure used to explain damage at the excavation boundary in high strength rock can be used to explain the onset and depth of mining-induced fractures in coal mine roadways once the impact of bedding is considered by assuming transverse isotropy. Based on this conclusion, the next sections of this paper seek to examine the collapse of ribs in coalmines, with particular reference to Australian underground coalmines.

GROUND SUPPORT

Kaiser and Cai (2014) discuss support for burst-prone ground in the context of sudden volume expansion during strain bursting. For strain bursting, support selection proceeds by estimating the depth of failure and the consequent bulking of the failed rock. The volume of the failed zone and the rock density is used to give the load demand on the support and the bulking provides the displacement demand. The displacement demand leads to selecting yielding support elements. Kaiser (2016) describes a gabion concept for large deformations utilising deep anchoring elements with good connections to a mesh/fibrecrete retention system for the material that will undergo brittle failure (Figure 4c). The idea behind this concept is that the gabion absorbs the energy released by strain bursting so that there is no kinetic energy left to cause ejection into the roadway. Kaiser (2017) proposes that the rate of brittle failure is possibly between 0.05 and 0.1 seconds and from this it is possible to determine a bulking velocity that can then be used in a calculation of energy that needs to be adsorbed by the gabion.

In Australian coalmines, the typical rib support that has evolved over the last three decades has similarities to the recently proposed gabion concept. It is suggested that the impetus for the typical Australian rib support has been the onset of brittle failure within the confined working area of the in-place miner-bolters. Bulking factors in coal ribs can be obtained from rib extensometry with values of 3% – 6% indicated in Error! Reference source not found.a for a light support pattern. It is speculated that the difficulties in achieving full encapsulation of rib bolts within the zone of brittle failure could be a positive result as it gives a displacement capacity that is required for any later bulking.

KINEMATICS AND SEISMIC SHAKEDOWN

The “burst” terminology used in coalmines presupposes violent ejection of coal from the sides of the excavation. Equally hazardous could be the simple collapse under gravity of large volumes of coal in mechanisms analogous to the collapse of excavation trenches in civil construction. Recent fatal collapses of ribs in Australian mines may be better explained by kinematics rather than bursting and it is for this reason that the title of this paper refers to sudden collapse. The kinematic hazard may not be as great a hazard in metalliferous mines due to non-persistence of joints, the use of drill and blast generating overbreak if joint blocks are present, and support installation that is not conducted in such confined spaces as on a continuous miner.

Undisturbed coal seams are typically characterised by two sets of persistent joints that are aligned orthogonal to bedding. For flat-lying seams, and hence sub-vertical joints, any planar slides or wedge hazards in the sides of underground roadways should be relatively small in
size but they may become hazardous given the confined work places; the toppling hazard would always be present. A light bolting density should easily control these hazards in flat-lying unfaul\ticker\nt coal seams. Any collapse of unbolted ribs, such as in front of a continuous miner, would be seen as sudden and since there would be some shearing through the roughness of joint surfaces some noise and dust would be generated.

When traversing fault zones, or even isolated small-throw faults, there may be persistent non-vertical joints that define wedges or planar slides with dimensions such that standard bolt lengths do not provide anchorage in stable ground (Figures 5a,b,c). The physical constraints on the continuous miners used in Australia mean that rib bolts are typically limited to a maximum length of about 2 m. For planar slides, joint dipping at less than about 50° would start being of concern in terms of suitable anchorage being available for practical bolt lengths; for wedges the concern would extend to possibly less than 55°.

![Planar slide prior to bolting](a) ![Wedge geometry in a faulted block of coal in a surface mine](b) ![Wedge defined by a joint dipping at 45°](c)

![Wedge is wider than the rib bolts are long](d) ![Decrease in stability of a wedge as a function of seismic loading](e)

**Figure 5: Kinematics of a coalmine rib**

The application of kinematics to rib collapse may provide a framework to better appreciate seismic shakedown and seismically triggered strain bursts. In rock slope design, the impacts of seismic events are assessed by invoking an additional acceleration to that of gravity. Figure 5e shows how the factor of safety of a wedge decreases from 1.2 (often interpreted as “stable”) to less than unity with an acceleration of 0.1g which is only a moderate Mercalli event according to Table 1. Extending these concepts further, there is a need to recognise that such low accelerations may be sufficient to cause the collapse of a slab of coal formed by earlier brittle failure if it had not been supported.

**SUMMARY AND CONCLUSIONS**

It is suggested that similar brittle failure and bursting mechanisms are present in both coal and metal mines and that the selection of support elements can use the same engineering concepts. Coalmines are potentially exposed to an additional sudden collapse mechanism in the form of wedge, planar and toppling failures of the type that are invoked in rock slope
engineering (Figure 6). Just as for strain bursts, kinematic collapse may be self-initiated (low static factor of safety), mining-induced (additional displacements during subsequent mining applying an additional vertical loads), or seismically triggered. It is recommended that workplace safety considerations are formulated in the context of sudden collapse of ribs instead of reference to “bursting” which is often used to include a component of ejection. It is suggested that this approach would concentrate attention on the critical importance of developing ground control strategies and less on the emotional aspects on mine seismicity, high stresses, and high velocity ejection.

The routine installation of bolts and mesh installed from the continuous miners and hence close to the development face in Australian coalmines provides adequate management for strain bursts. Sudden failure of a coal rib may be encountered when the mining system is not compatible with the installation of bolts and mesh – for example longwall extraction – but the resulting hazards should be able to be managed with stand-off distances. In the restricted workspace of the miner-bolters used in Australia, kinematic failures represent a significant hazard, which in some situations may not be controlled with current support patterns.

![Figure 6: Sudden rib movements can be induced by kinematic collapse, strainbursts and gas outbursts.](image)

<table>
<thead>
<tr>
<th>Type</th>
<th>Sub-set</th>
<th>Energy source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal burst</td>
<td>Strain burst</td>
<td>Strain energy at boundary</td>
</tr>
<tr>
<td></td>
<td>Fault slip burst</td>
<td>Tectonic strain</td>
</tr>
<tr>
<td></td>
<td>Pillar burst</td>
<td>Overburden gravitational</td>
</tr>
<tr>
<td>Outburst</td>
<td></td>
<td>Gas</td>
</tr>
<tr>
<td>Kinematic collapse</td>
<td>Wedge, planar, topple</td>
<td>Seam level gravitational</td>
</tr>
</tbody>
</table>

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NUMERICAL MODEL OF COAL BURST MECHANISMS

Gaetano Venticinque¹ and Jan Nemcik²

ABSTRACT: Coal bursts present one of the most severe hazards for the underground coal mining industry. In Australia, coal burst events are becoming increasingly frequent as coal measures are mined progressively deeper. Until now, coal burst mechanisms haven’t been properly understood. A significant ongoing effort and a large number of research activities are searching for answers. This work is supported by the Australian Coal Association Research Program (ACARP) which aims to provide explanation of the probable mechanisms and key factors behind the coal burst phenomena. The available energy required to eject the coal rib into the mine opening has seeded the idea of momentum transfer from within the seam towards the rib side. This mechanism has a strong analogy to Newton’s Cradle device and hence conservation of momentum and energy principles that can account for momentum transfer between confined seam masses at a distance and ejected unconfined fractured mass at the free surface of the rib. Using dynamic analysis, preliminary numerical models successfully simulate fast ejection speeds of coal rib material and thus identify a probable common cause of coal bursts in mine roadways. Modelled coal mine roadway in 3 m thick seam at a depth of 550 m successfully simulated coal burst phenomena; laterally ejecting 3.92 tonnes of coal from the rib with velocities ranging up to 2.3 m/s. Recognising that ejection speeds are dependent on material properties, extent of trigger induced failure between coal/rock boundary and chosen geometry; a few modelled cases are presented here.

INTRODUCTION

Several coal burst occurrences with loss of life in underground Australian coal mines have prompted the government inspectorate and coal mining industry authorities to devise safer working methods of mining deeper coal deposits. Coal bursts are very difficult to predict as they are inherently not frequent, isolated and occur without warning. To minimise the occurrence of coal bursts it is necessary to first understand how the mechanism of coal bursts arises. Up to now this remains elusive with many researchers investigating different combination of stresses, mining geometries, fault locations and other factors to predict coal burst occurrence Hebblewhite and Galvin (2017), Dou, et el., (2016), Bräuner (1994), Mark and Gauna (2016), Moodie and Anderson (2011), Calleja and Porter (2016). Many numerical attempts have proven unsuccessful; largely due to unsuitable methods or programs employed. Consequently existing models have been unable to explain the coal burst mechanisms satisfactorily Chengguo and Canbulat(2017), Muller (1991) and others. Frequent mis-use of conventional elastic-plastic, strain softening and hence otherwise static based models are attributed towards significantly limiting both theoretical derivation and computational ability in analysing fast dynamically occurring events. This highlights the serious shortcomings of trying to model dynamic material response behaviour of sedimentary or igneous rock strata around excavations; hence such models should not be used. The importance of built in dynamics is therefore recognised in dynamic analysis for enabling real time simulation of dynamic ground movement. Likewise, when using these models, correct approach is necessary to observe what mechanisms are taking place.

Supported by ACARP, this project is focused on computational systems to mimic natural ground dynamics and model possible types of dynamic events that simulate the coal burst process. At this stage only simple, reproducible models were chosen with various parameters and geometries to achieve coal ejection from the rib side; supporting the concept of the energy transfer and conservation of momentum \( p = mv \) where \( m \) is the coal mass and \( v \) its velocity. Several parameters influence the coal rib ejection speeds. These include material properties, initiation of coal/rock interface failure, its extent and the propagation of failure. At this stage the modelled depth of cover of 550 m was kept similar to the Austar mine coal burst

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incident location. To simulate the dynamic coal burst event, coal-rock bond failure was trialled at various distances from the mine roadway. The aim of this model was to investigate whether coal burst events, such as that observed in Austar mine at 550 m depth can be reproduced via simulation.

**NUMERICAL MODELS OF COAL BURSTS**

Using the 2-dimensional Fast Lagrangian Analysis Continua (FLAC), (ITASCA 2015), an excavation of a standard underground coal mining roadway was modelled in a 3 m thick coal seam bound by a strong sandstone stratum at a depth of 550 m as shown in Figure 1. Initially, the model was brought to static equilibrium prior to dynamic solution being initiated (as described in FLAC 2015) while removing part of the coal-rock interface bond to evaluate the system response to coal-rock interface failure. The simplified strata properties are provided in Table 1 while the model geometry grid is shown in Figure 1. It is known that during dynamic fracture propagation, cohesion will drop to zero while the coefficient of friction can drop rapidly from its static value to a much smaller value. Brown (1998) noted from his experiments that during rapid sliding, the friction coefficient reduced by a factor of up to about seven times. Brown’s comparison with other experimental results suggests the reduction of normal stress by interface separation waves is the most likely explanation. Therefore for the purpose of this study only (proof of concept), the bond properties 1 m in length along the coal-rock boundary at the roof level were removed at various locations.

**Table 1: Modeled strata properties**

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Density (kg/m³)</th>
<th>Bulk Mod (MPa)</th>
<th>Shear Mod (MPa)</th>
<th>Friction (Degrees)</th>
<th>Cohesion (MPa)</th>
<th>Tension (MPa)</th>
<th>Dilation (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor rock</td>
<td>2,500</td>
<td>6.67e⁹</td>
<td>4e⁹</td>
<td>37</td>
<td>10e⁶</td>
<td>4e⁶</td>
<td>0</td>
</tr>
<tr>
<td>Coal</td>
<td>1,400</td>
<td>3.33e⁹</td>
<td>1.11e⁹</td>
<td>35</td>
<td>0.2e⁶</td>
<td>0.2e⁶</td>
<td>0</td>
</tr>
<tr>
<td>Roof rock</td>
<td>2,500</td>
<td>6.67e⁹</td>
<td>4e⁹</td>
<td>37</td>
<td>10e⁶</td>
<td>4e⁶</td>
<td>0</td>
</tr>
</tbody>
</table>

**Figure 1: (a) FLAC model grid, (b) Yielded rib of mine roadway and attached coal block**

Preliminary dynamic simulation trials clearly indicate that the ejection of a large coal mass from the rib side is only possible if the stored compressive energy within the pillar is released and accumulated as the compression wave travels towards the rib face. Stemming from the “Newtons Cradle” idea shown in Figure 2, this is only possible via the conservation of momentum that begins several metres into the pillar, propagating towards the unconfined rib side.
Figure 2: Modified Newtons Cradle - Analogy of the rock burst

For the energy trigger to occur, a dynamic event such as a fault slip is needed to generate seismic waves and break some of the already stressed bonds that exist between the coal seam and the rock surface. This failure needs to occur either at the top or bottom of the seam several metres away from the rib side. If loss of cohesion occurs simultaneously at both top and bottom of the seam, the coal burst may become more violent.

Figure 3: Dynamic response to coal/rock interface failure resulting in rapid coal expansion and ejection of the loose coal block at the rib boundary.
In this model a 1 m section of total bond failure between the coal and rock at the roof level only was initiated at various distances from the rib side. Once the bond was broken, the dynamic chain reaction took place where the released elastic compression energy was carried at great speeds across the seam towards the rib side. The ejected coal rib is usually fractured and unconfined, for simplicity this was modelled as a 1 m thick loose block of coal 3.92 tonnes in weight as shown in Figure 3 (a). The dynamically generated mass velocities that propagated towards the rib side propelled the loose coal block in the lateral direction at up to 2.3 m/s depending on the location of failure. This momentum transfer in the model can be clearly observed in Figure 3 (b, c, d, e and f). Prior to the ejection, velocities within the seam are high. Once the coal block is ejected, the velocities in the seam decay to negligible levels as the accumulated energy from the seam is transferred into the kinetic motion of the detached block shown in Figure 3(f). The modelled displacements shown in Figure 4 indicate the triangular shape disturbance that starts at the beginning of the disturbed zone and gets wider towards the rib side. This disturbed shape is similar to the ejected coal cavity observed after the coal burst event at the Austar mine. This mechanism further reinforces the findings that the coal-rock bond failure occurred along the “Dosco parting” which is the dominant bedding plane located at the roof level in the Austar mine.

To study how the rib block ejection velocities vary with the coal-rock bond failure, various locations of cohesive/frictional bond loss 1 m in length between the roof rock and coal seam were artificially induced. In the first model, the edge of 1m de-bonded roof length was placed 1 m from the rib (rib located at 15 m). In all subsequent models the disturbance was gradually shifted in 0.5 m intervals further away from the rib. Altogether nine cases were modelled. The ejected velocities of the 3.92 tonne block were graphed and are presented in Figure 5 (a).

**Figure 4:** (a) Modelled displacements within the coal seam after coal ejection indicate the triangular shape of seam disturbance (b) Austar coal burst ejection along the Dosco Parting (Photograph after Australian Mine Safety Journal, 2016)

**Figure 5:** (a) Modelled location of the 1 m long coal-roof bond loss versus the ejected velocities of the 1 m thick rib coal block. (b and c) Vertical and horizontal stress profile in the coal at the roof level adjacent to the coal rib before the rock burst occurred.
The results show that the most violent coal burst occurred when the 1 m long bond failure along the roof occurred between 3 and 4 m from the rib side. This failed zone (at the chainage between 11 and 12 m in the model) coincided with locations where the maximum vertical and horizontal stresses were on the steep decline towards the rib side as shown in Figure 5 (b and c).

POSSIBLE COAL BURST TRIGGER MECHANISMS

Many coal burst trigger events may exist such as: failure of a highly loaded fault plane, geologically weakened coal-rock interfaces, on high additional loads due to nearby mining. As the nearby fault planes were present adjacent to the coal burst location in the Austar mine (Figure 6.) it is assumed that their failure may have generated enough seismic energy needed to disturb the Dosco coal parting strength and trigger the coal burst.

Figure 6: Location of faults at close proximity to the coal burst site in Austar mine
(Hebblewhite, 2017)

Mining activities may have unloaded some of the compressive stress normal to the fault plane causing the rapid slip movement along the fault. The rapid release of seismic energy propagates as a compressive wave through the solid strata that can provide enough energy to trigger the coal burst.

It is worthwhile to discuss the attenuation of seismic energy from the fault plane source. From basic physics (neglecting the effects of damping over short distances), when considering the point energy source in three dimensions, seismic energy through the elastic medium attenuates with a distance squared as shown in Figure 7 (a), whilst for sources of energy generated along a long line, attenuation with distance is linear as shown in Figure 7 (b). When the energy source is a large fault plane area and within a relatively short distance that is smaller than the fault geometry, the energy does not decrease with the distance as illustrated in Figure 7 (c).
Figure 7: Aerial expansion of the seismic energy in 3-dimensional space from (a) the point source (b) the line source and (c) the plane source

A simple numerical model of the fault slip oriented at approximately 45° dip towards the left and approximately 10 m from the roadway rib side, is depicted in Figure 8. The compressive waves refracted through a coal seam (shown as grid velocity waves) indicate how a violent behaviour can easily disturb the already highly loaded coal-rock bonds. Note that the waves propagate faster in rock and slower in coal due to the different material stiffness. This creates refractions and interference patterns at the coal-rock interfaces.

Figure 8: A simple simulation of fault slip adjacent to the mine roadway demonstrates an example of fast compressive waves presented as grid velocities movement away from the fault zone.

CONCLUSIONS
The main purpose of this research is to explain the concept of the coal burst mechanism in a coal mine roadway. The initial idea originated from the Newton’s Cradle, where the energy transfer occurs via the conservation of momentum principle. The dynamic numerical FLAC model proved suitable in demonstrating this concept. The aim of this work was to approximate the coal burst event that occurred in the Austar mine, Australia. In a nutshell, the model clearly demonstrated the release of stored potential energy (compression) within the seam and its conversion into kinetic movement of the unconfined/loose coal mass. A large portion of this energy was transferred via the conservation of momentum into the 3.92 tonne coal block located at the free surface, ejecting it at velocities of up to 2.3 m/s. After the coal block was ejected the modelled displacements within the rib indicated a triangular deformation mode of failure. This type of failure and its location was similar to the Austar mine coal burst failure geometry. It can therefore be concluded that the Dosco roof parting failure in Austar mine caused ejection of the triangular portion from the upper seam section to a depth of approximately 4 m.

At this stage of research presented here, the main aim is to explain the mechanism of coal burst adjacent to the mine roadway. Material properties, geometry of mine workings,
geological disturbances, level of stress and other factors that may influence the likelihood of coal bursts are yet to be thoroughly investigated in detail. As part of the follow-up work more accurate models are being prepared to enable simulations of other coal burst cases, eventually leading to better understanding and prediction of these events. Mathematical calculations of the energies locked in the seam and the energy release have been pursued in parallel to provide better prediction methods against coal/rock bursts.

Many different types of coal bursts have been experienced mainly in deep overseas mines. As the Australian mines are getting into deeper ground all styles of coal/rock burst need to be investigated. A similar dynamic analysis approach will need to be undertaken in order to establish potential variations of coal burst mechanisms and search for methods that can minimise their hazardous occurrence.

REFERENCES
INSIGHTS INTO THE ENERGY SOURCES OF BURSTS IN COAL MINES AND THE EFFECTIVENESS OF PREVENTION AND CONTROL MEASURES

Mahdi Zoorabadi¹ and Winton Gale²

ABSTRACT: Coalburst is a general term, which is commonly used in the coal mining industry for the violent failures of coal in the ribs and face of roadways and panels in underground coalmines. Due to lack of interest in the industry to reveal the causing source of the event, or due to uncertainty about the source, they happily use this term. The term by its own does not reveal the source of the energy, which causes the event. There are three sources of energy that can cause a burst event in underground coalmines: 1) store elastic strain energy, 2) seismic events and 3) gas expansion energy. This paper presents the fundamentals about these sources of energies and discusses our known and unknown facts about the mechanisms. Additionally, it discusses the reliability and effectiveness of stress relief holes and gas exhaust holes as controlling measures to prevent burst events.

INTRODUCTION

Bump, pressure bump, pressure burst, coalburst and outburst are the terms which have been used to describe the events in underground coal mines which cause audible sound, ground vibration, violent failure of coal, and propulsion of coal and gas from the ribs or working face. The scale of burst events varies from very small coal spitting from a roadway’s face to dislodgement and propulsion of hundreds tonnes of coal from the face or ribs. Apart from the source and mechanism of the burst events, they are considered a significant risk for workers and the financial success of an underground coal mine.

Recently ACARP provided several research funds through the coalburst task group to study the mechanism of the burst events in underground coalmines (Elvy 2015). SCT Operations Pty Ltd secured two funds as follows:

- Mechanics of gas related bursts in mining (C26060).
- Energy, burst mechanics required for coal bursts and energy release mechanics (C26066).

This paper presents an insight into the available energy sources to cause a burst event. It also discusses the effectiveness of the gas exhaust holes and stress release holes as two common preventing control measures.

AVAILABLE ENERGY SOURCES IN UNDERGROUND COAL MINES

When a burst event occurs, the broken coal from the ribs or face is propelled into the roadway or into the longwall working space. Figure 1 shows an example of a burst event in which the coal blocks were propelled from the roadway face and landed at distance around 4 m behind the face. If it is assumed that the blocks sat on the face as a free-body (with no resistance), the trajectory graphs of the block for various initial velocity can be calculated as Figure 2.

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Figure 1: Laser scanner image of a burst zone

Figure 2: Displacement and trajectory of a free coal block sitting 2.5 m above floor on the roadway face with various initial velocities applied

The Figure 2 shows that it takes only 0.71 s for a free block of coal to hit the roadway floor if it is propelled from 2.5 m above the floor. The horizontal distance that the block travels depends on its initial velocity (velocity of burst). For a block to travel 4 m before hitting the floor, the initial velocity should be around 6 m/s. As previously mentioned, this velocity is required for a block of coal with no resistance. However the coal in the roadway face and ribsides carries the redistributed vertical stress which provides considerable confinement and resistance. Figure 3 shows 3D numerical model results of the vertical stress distribution at the face and right rib of a roadway excavated in Bulli seam geology with the maximum horizontal stress is acting perpendicular to the roadway. Considering this fact, the available energy of the burst is required to firstly overcome the resistance provided by confinement (frictional resistance) then new fracturing needs to be generated to free the coal blocks. Finally, the remaining energy will turn into the kinematic energy, which controls the initial velocity of the blocks and propel them.
There are three sources of energy with the potential to cause burst event in underground coal mines; 1) stored elastic strain energy, 2) seismic energy, 3) gas expansion energy (Gray 1983; Gale 2002), which are explained in the following sections.

**Stored elastic strain energy**

When a sample of rock or coal is loaded under uniaxial stress condition, the external work done by the loading machine is stored in the sample while it deforms due to the applied stress. This energy is equal to the area under the stress-strain curve (Figure 4) and is calculated as follows and shown in Equation 1 (Jaeger and Cook, 1979):

\[
\text{Elastic Energy (} W_e \text{)} = \frac{1}{2} \sigma \varepsilon \quad \sigma = \varepsilon E
\]

\[
W_e = \frac{1}{2} \frac{\sigma^2}{E} \quad (\text{MJ/m}^3) \quad (1)
\]

The energy stored in the rock unit and coal is related to the applied stress and the stiffness of the materials. The rock can be visualised 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. For a unit volume of coal under in situ stress conditions, Equation 2 forms the following formula:

\[
\text{Elastic Energy} = \frac{1}{2E} \left[ (\sigma_1^2 + \sigma_2^2 + \sigma_3^2) - 2v(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1) \right]
\]

\[
(2)
\]

where \(E\) presents the elastic modulus of the rock or coal, \(v\) is the poisons ratio, and \(\sigma_1, \sigma_2, \sigma_3\) are the principal stress components. These equations show that for a constant stress fields, the magnitude of the energy stored in a softer sample is more than in a stiffer sample due to the increased deformation that occurs in the soft sample. On the other hand, the softer rocks such as coal, attracts lower tectonic stress compared to stiffer rocks, which can reduce the stored energy.
Equation (2) was applied to a 3D elastic model of the roadways excavated in Bulli seam geology. The stress conditions were related to a depth of 500 m, maximum horizontal stress was related to a Tectonic Stress Factor (TSF) of 1.6 and the minimum horizontal stress was related to a TSF of 1. The stored elastic energy in coal obtained from this 3D modelling is presented in Figure 5. For the modelled stress conditions, the stored elastic energy in the coal ribsides is approximately 20 KJ/m³ and it increases to a maximum of 45 KJ/m³ at intersections where stress concentration occurs.

In addition to the elastic energy, which is stored in a unit volume of coal, additional energy is available from unloading of the roof and floor or surrounding coal in the failure process. The generalised concept is presented in Figure 6, which relates to the surrounding strata (or coal in a thick seam) as a spring which when unloaded by the failure of the coal, can rebound and provide additional energy to the system. This energy is commonly required to form macro fractures in the coal, however depending on conditions additional energy may be available which could contribute to the energy level required to propel the fractured coal.
Figure 6: Concept of additional stored elastic energy

An approach that takes into account the failure stiffness of the coal can be used to calculate the additional stored elastic energy as follows:

\[ W_{ae} = \frac{\text{stress drop}^2}{2(\varepsilon_r^0 - \varepsilon_c^0)} \]

where the stress drop is that at the time of failure of the coal, \( E_r^0 \) is the unloading modulus of the surrounding rock and \( E_c^0 \) is the post failure modulus of the coal (or rock units undergoing fracture). All units are in MPa and energy is in MJ/m³.

This energy can contribute to the propulsion of the outside skin of a rib and face. The initiated burst process can grow further into the rib or face if the additional energy is sufficient. Since the magnitude of the stored elastic energy and available energy from the unloading of the surrounding strata are not significant, the scale of the burst initiated by them is small.

Seismic energy

The seismic events that affect mining activities can be grouped into mine induced seismic events and regional seismic events. The regional seismic events are generated due to regional tectonic activity and where mining production has no contribution to the occurrence of them. This paper discusses only the mine induced seismic events, which are directly initiated by the mining activities.

The initial stress in the ground is disturbed by excavation of roadways or retreating of longwall panels. When the magnitude of the new distributed stress exceeds the tensile or shear strength of the intact strata, the strata will fail and a portion of the energy stored in the volume of the failed strata is released in the form of seismic events. Additionally the redistributed stress can exceed the strength of the bedding planes, pre-existing fractures and faults. In these cases, the energy, which was stored across the rupture plan is also released and causes seismic events.

The energy released by a seismic event depends on the amount of stress drop, average displacement along the fracture, and the surface area of the stress induced fracture (Mendecki 2013). When rock fails, the difference between the initial stress to the residual stress presents the stress drop. As shown in Figure 7, the stress drop associated with tensile fracture is significantly less than shear failure. Also, for the shear failure formed in the tensile side of the strength curve (Zoorabadi et al. 2017) the stress drop can be significantly less if the minimum stress is compressive.
Considering the smaller shear and tensile strength for bedding planes and pre-existing structures such as faults compared to the intact rock, the stress drop associated with the shear or tensile failure of them will be smaller. Despite this, the surface area and displacement for pre-existing structures could be significantly higher.

The amount of energy released through forming a square shape shear fracture within an elastic medium is calculated by the following equation (Gale 2002):

\[ E_{\text{Energy}} = \frac{\Delta \tau^2 \times L_f^3}{2G} \]  

(4)

where, \( \Delta \tau \) is shear stress drop, \( L_f \) is the fracture length, and \( G \) presents the shear modulus of the rock.

The energy associated with a tension fracture is estimated as:

\[ E_{\text{Energy}} = \frac{\Delta \sigma^2 \times L_f^3}{E} \]  

(5)

where \( \Delta \sigma \) presents the magnitude of the stress drop due to tensile fracturing and \( E \) is the elastic modulus of the rock.

Several empirical formulations have been introduced by seismologist to calculate the energy of a seismic event on the basis of its magnitude. For this paper the following equation was selected (Denton P., 2014):

\[ \log (E_{\text{Energy}}) = 4.8 + 1.5M \]  

(6)

where \( M \) is the magnitude of the seismic event in Richter, and energy of seismic event is in joules.

Equations 4 to 6 were used to calculate the approximate fracture length associated with various magnitudes of the seismic events as presented in Figure 8. These results show that the magnitude of the stress drop, which is expected to be higher for shear failure under compressive confinement, has a significant impact on the energy and magnitude of the induced seismic event.
As previously mentioned, seismic events are induced by the sudden release of stored elastic energy throughout rock fracturing or reactivation of pre-existing structures which occur in surrounding strata. The mine induced seismic events are very common in both hard rock mining and coal mining. In coal mining, coal bump is a commonly used term which has been used for describe the events that generates audible sounds and ground vibration. In fact, the bumps are seismic events and they can dislodge fractured coals from the ribs or face into the working areas. Despite the limited number of publicly reported burst events induced by seismic events in coal mines, coal bumps have been regularly reported by deputies and operators in most of the underground coal mines. Therefore in contrast to the common view that coalburst induced by seismic events are very rare and isolated, they are occurring regularly in the coal mines.

A seismic monitoring network is commonly used in underground mines to locate the seismic events and measure the source parameters such as radiated energy, mechanism, corner frequency, magnitude, stress drop, etc. (Figure 9).

Figure 8: Fracture size versus magnitude of seismic event for various fracture modes

Peak Particle Velocity (PPV) is a commonly measured parameter, which is reported for each seismic event. The PPV is a function of seismic event magnitude (energy), distance between measuring point and seismic event location, and wave deterioration through wave travel from source to the measuring point. Kaiser et al. (1992) proposed the following equation to

\[
PPV = \frac{E}{r^2} \cdot \frac{1}{S} \cdot \frac{1}{d} \cdot \frac{1}{T}
\]
estimate the PPV from event magnitude on the basis of their experience from hard rock mining in Canada.

\[ PPV = 1.4 \frac{10^{M_r/2}}{r} \]  \hspace{1cm} (7)

where PPV is in m/s, \( r \) is in m and \( M_r \) is the magnitude of seismic event. Kaiser et al. (1993) proposed PPV less than 50 mm/s as the threshold for no damage, falls of loose rock occur at 50 mm/s <PPV< 300 m/s, falls of ground for 300 mm/s <PPV< 600 mm/s and severe damages are expected to occur at PPV> 600 m/s. These thresholds are not applicable for all conditions and the induced damage depends on rock mass strength and characterisations of support system. The literature from hard rock mining cases (McGarr et al., 1981; Milev et al., 1999) show that when a seismic event reaches to the fractured rocks around the roadway, the PPV can be amplified up to 10 times compared with PPV measured in intact rock (Figure 10). The frequency of the seismic wave and thickness of the fractured zone control the amount of PPV amplification.

SCT Operations database for seismic event occurring in development show that the average event magnitude is in the range of -2.7 to -2. The general data appears to have a normal distribution relationship with a largest expected magnitude of approximately -1.25. It is also noted that there are a small number of events which were in the -1 to -0.25 range.

![Figure 10: Concept of PPV amplification in broken ground](image)

When a roadway is approaching a fault zone, the fractured and softened zone ahead of the face may push the stress concentration into the coal ahead of the face. Additionally the stiffness of the ground in a fault zones is usually lower than in surrounding ground. Therefore the redistributed stresses ahead of a roadway face may not transfer through fault zones as easily and may become more concentrated on one side of the fault zone (Figure 11). This stress concentration creates a shear stress on faults zones and can exceed the shear strength of the fault close to the roadway. If fault is located above or below a roadway, the stress release toward roadway reduces the normal stress acting on a fault which results in a drop in shear strength. Both mechanisms may induce a shear slip along the fault zone. In addition to this, the stress concentration ahead of the face caused by approaching a fault zone increases the rock fracturing and corresponding seismicity.
Figure 11: Stress concentration ahead of roadway face approaching a fault zone

Alternatively, considering the strain in the ground associated with the fault zone and its en-echelon nature, the stress in the fault zones may be elevated above the normal ground conditions. This is due to the start-stop nature of the fault and their varying throw along strike. Such a variation is likely to disturb the normal stress regime. Considering the possibilities to have elevated stress about faults, more seismic activity with relatively larger magnitude are possible for development approaching a fault zone such as the fault zones.

The mine induced seismic events through longwall retreat could have much higher magnitude compared with the roadway development. The average event magnitude for the longwall retreat varies between -1.5 to -1 with an upper expected of 0.5. However there is potential for inducing large events with magnitude 2-3.5 Richter for longwall retreat.

Figure 12 shows the PPV induced by seismic events with magnitude between -2 to 0.5 at various distance from the event location. Therefore a seismic event (or bump) with magnitude of 0.5 occurring at 5 m ahead of face is able to generate a PPV of 498 mm/s at the face. This PPV is has potential to cause a minor burst and fall of ground.

Figure 12: PPV induced by seismic events at various distance from event location
Gas expansion energy

Gases in coal are divided into two groups; 1) free gas, which exists in the pores and open cracks and forms only a small percentage (approximately 5-10%) of the total gas and 2) adsorbed gas. Most of the gas in coal is present in a sorbed phase on the internal surface of coal. The coal molecules are attached to these internal surfaces (pore space and cracks) as mono- or multi-layer. Since coal has very large internal surface area, it has high gas adsorption capacity.

The free gas is a function of coal porosity and gas pressure can be estimated by the following equation:

\[ Q_{\text{free gas}} = \frac{273nP}{TP_0} \]  

where, \( n \) is the porosity of coal, \( P \) is the gas pressure (KPa), \( T \) is strata temperature in degrees f Kelvin, and \( P_0 \) is the atmosphere pressure.

Coal with porosity of 10% and temperature of 20 degrees of Celsius, can have 1.38 \( m^3/m^3 \) of gas as free gas under a gas pressure of 1.5 MPa. The relation between pressure and adsorbed gas for a constant temperature is called isotherms and is presented by Langmuir's equation as follows:

\[ Q = \frac{abP}{1+bp} \]  

where, \( Q \) is the volume \( (m^3/t) \) of the gas adsorbed at a given pressure of \( p \) with dimension of bars, \( a \) is the Langmuir's constant representing the volume of the adsorbed gas \( (m^3/t) \) when pressure approaches to an infinite value, and \( b \) presents the Langmuir's constant with dimensions of 1/bars.

The amount of adsorbed gas depends on the rank of coal, gas type, pressure, moisture content, ash content, temperature. The gas absorption capacity of coal for carbon dioxide is almost 2.5 to 4 times that methane. Both stress and gas play a role in the process of the appearance of coal burst events involving gas expansion. The burst process is conceived as occurring in eight stages as follows:

1. In situ stage: Coal in front of the face is under triaxial stress condition and its strength is a function of the applied confined stress.
2. As the face approaches an area with high gas content (gas pocket) the water pressure reduces and the confining stress can reduce.
3. If the water pressure drops below the desorption threshold, then pore space and existing fractures (micro and macro) can fill with gas.
4. If the pore pressure is greater than the confining stress and strength of the coal then propagation of fractures and new fractures can form.
5. This allows more gas to be desorbed from the coal matrix and a greater volume of gas to be readily available to contribute to a burst.
6. If the volume of desorbed gas is higher that the volume of gas that can flow to the roadway then elevated pore pressures remain.
7. If the pressure developed is sufficient to push the coal from the ribside then a burst occurs. The strength of the rib is a control on the pressures required to initiate a burst.
8. Once a rib burst has been initiated the expanding gas acts as a means of maintaining energy to the coal being expelled.

In the final stage, all fractured coal is ejected from the gas pocket. The fracturing process can be propagated into surrounding coal but those areas remain in place and cannot be ejected. The fracturing of the surrounding coal increases gas desorption which is recorded as a spike in gas emission data. Gas expansion energy for adiabatic expansion for a pocket of gas with pressure of \( P_1 \) to a new pressure of \( P_2 \) (atmosphere pressure=101 KPa) is calculated as follows:
\[
W_{\text{gas}} = \frac{P_1 V_1}{\gamma-1} V_1^{1-\gamma} (1 - \left(\frac{P_2}{P_1}\right)^{\frac{\gamma-1}{\gamma}})
\]  
(10)

where, \(W_{\text{gas}}\) is the expansion energy of gas (J), \(P_1\) is the initial absolute pressure (Pa), \(P_2\) is the final absolute pressure (Pa), \(V_1\) is the initial volume of gas, \(\gamma\) is approximately 1.3 for methane and carbon dioxide.

The above mentioned mechanism for the coal burst induced by gas expansion occurs in a short time which requires a considerable portion of the adsorbed gas being desorbed during that short time. The available gas expansion energy for 1 tonne of coal with gas content of 10 m\(^3\)/tonne and porosity of 10% for various desorption scenarios is presented in Table 1.

The available energy from gas expansion compared with other source of energy is higher if the desorption rate is high enough or if the volume of the free gas in pore space is significant. The probability of having these conditions is higher in the area close to geological structures. For example for mylonite in a fault zone, the volume of free gas is higher due to higher porosity and desorption rate is also higher because of smaller particle size. In addition to this, a barrier with low permeability is required to help gas pressure build up ahead of the face or in ribsides. Without this barrier the gas can drain through fractured coal with no pressure build up which is a main requirement for the gas expansion energy.

<table>
<thead>
<tr>
<th>Portion of desorbed gas to total gas [%]</th>
<th>Available gas expansion energy from free and desorbed gas (For gas pressure 1.5 MPa) [KJ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>388</td>
</tr>
<tr>
<td>15</td>
<td>806</td>
</tr>
<tr>
<td>30</td>
<td>1433</td>
</tr>
<tr>
<td>45</td>
<td>2060</td>
</tr>
</tbody>
</table>

EFFECTIVENESS OF BURST CONTROL MEASURES

Methods to control burst events during development includes stress relief boreholes, gas exhaust boreholes, cable bolting, physical barrier on the miner, and remote mining. The effectiveness of stress relief holes and gas exhaust boreholes are discussed in this paper.

Stress relief holes

Stress relief boreholes are used in a number of countries as a means of minimising the potential of coal bursts in high stress conditions. Boreholes act similar to excavations in rock and they disturb stresses and impose a new stress distribution. If the stress around the borehole exceeds the strength of coal, a fractured zone is formed. The broken coal will reduce stress based on its residual strength characteristics. Therefore, if fracturing occurs there will be some stress relief about the borehole. If fracturing does not occur, then there is no vertical stress relief adjacent to the borehole, rather an increase in vertical stress.

This process was investigated through numerical modelling. In the modelling, a borehole with diameter of 50 mm is drilled in coal with Bulli seam geotechnical properties at a depth 500 m and tectonic stress factor (TSF) of 0.5, 1, and 1.6. The principal stress if vertical and the horizontal stress is varied on the basis of tectonic stress factors. Figure 13 show the failure modes and extent of the failure zones for each TSF value. Shear failure and tensile failure and reactivation of these are the failure modes of the fractures that form around the borehole due to stress concentration.

Despite the different fractured zone extent for various TSF values, the fractured zones only extend 2R (R is the radius of the borehole). Therefore the stress released zone around the each borehole is limited to only two times of the borehole radius. These results show that a large number of closely spaced boreholes would need to be drilled in a roadway face to
release stress concentration ahead of the face. These results are in accordance with drilling practice and experience in creating a stress relief slot about an excavation.

Gas exhaust boreholes

Gas exhaust boreholes are drilled into the roadway face to act as high conductive pathways for desorbed gas to be drained from the face without building pressure in pore spaces. This method has been used for several decades in Europe and Australia, but its efficiency is open to question. There is mixed experience with exhaust holes, which is most probably due to the variation in desorption rate and location of the gas source for the cases where it has been used.

The efficiency of gas exhaust borehole was investigated using 3D numerical modelling. For this model, a gas pocket was assumed to exist 1.5 m into the face of a roadway developed in Bulli Seam geotechnical conditions. A typical adsorption isotherm for Bulli coal was used for this modelling. For the purpose of this assessment the desorption isotherm is assumed to be the same as the adsorption isotherm. Gas content was assumed to be 10 m$^3$ which is equivalent to 1.5 MPa gas pressure according to this isotherm.

The gas desorption mechanism was implemented in a 3D numerical model of the roadway, where it was assumed that only 20% of adsorbed gas is able to desorb at early stages of desorption. Figure 14 shows the burst caused at the face due to gas pressure. This analysis shows that a gas pressure of 1.5 MPa is sufficient to burst the roadway face under modelled seam geotechnical conditions. This is not considered to be the pressure threshold for a burst but a value used for the assessment.

The efficiency of the gas exhaust boreholes to eliminate or downgrade the severity of a coal burst was investigated by modelling five boreholes drilled into the face with approximate spacing of 1 m. The boreholes locations are presented in Figure 15a. The results were not significantly different from the previous model, where the gas pocket is located 1.5m behind the face and gas content is 10m$^3$/ton. The gas flow is shown in Figure 15b and shows that although pore pressure along the exhaust borehole has dropped significantly, their efficiency to take all desorbed gas out of the face is compromised by the ability of the gas to transport between holes.

Figure 13: Damage and stress relief zone about a borehole under a variety of horizontal stress conditions
The conductivity of the fractures intersecting boreholes is very important in allowing gas to migrate in a pre-burst time zone, but in the instance of a burst the gas migration is too slow to allow the boreholes to exhaust the gas. The modelling indicates that if gas desorption or diffusion takes place at a modest rate, then boreholes can drain gas and minimise pressure build-up. If the volume of free gas is the main driver of gas expansion energy (Mylonite zone), the gas exhaust hole can drain the free gas ahead of the face. However, if gas desorption occurs at a very fast rate (which is typical of bursts), then such holes may not move a sufficient volume of gas in time.

CONCLUSIONS

Bursts events in underground coalmines are risks for the safety of workers and the financial success of projects. Although the Australian coalmine industry previously used different terms such as bump, pressure bump, coalburst, and outburst to report the events, but the term coalburst is becoming the commonly used term in recent years. This term on its own does not reveal the mechanism of the burst and the driven source of the energy. SCT Operations managed two ACARP projects to investigate the energy and mechanism of bursts. This paper discussed the available sources of energy for burst events in underground coalmines. The stored elastic energy, seismic energy and gas expansion energy were discussed and the methods to quantify the available energies were presented. Research on this topic is not yet
complete and so this paper presents the current understanding and knowledge obtained through these research projects.

In addition to this, the effectiveness of stress relief holes and gas exhaust borehole and control measures were discussed in this paper. The 3D modelling showed that the stress relief holes only release the stress in volume of surrounding coal within two times the hole radius. Therefore a large number of closely spaced boreholes would need to be drilled at the roadway face to release stress concentration ahead of the face.

The 3D modelling of gas exhaust boreholes also showed that the exhaust holes are only effective where the conductivity of coal is high enough to allow the gas to drain quickly through holes.

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ANALYSIS OF BULLI SEAM BENCHMARK AND DRI TO DETERMINE OUTBURST THRESHOLD LIMITS

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ABSTRACT: Following the introduction of the Desorption Rate Index (DRI) and Bulli Seam Benchmark to the Australian coal industry in 1995, the use of the DRI\textsuperscript{900} method was adopted and continues to be used as the primary method to assess outburst risk and to determine gas content threshold values for outburst risk management in Australian coal seams. In addition to assessing outburst risk based on gas content threshold values, several Australian coalmines also include DRI\textsuperscript{900} as a threshold level in assessing outburst risk. It is apparent that there is a broad lack of awareness and understanding of the limitations and deficiencies of using DRI to assess outburst risk and to determine appropriate outburst threshold limit values.

A comprehensive set of gas test results from Australian coal seams has been collected as part of research into control and management of outburst risk in Australian underground mines and the results of specific investigation into DRI and its applicability for use in assessing outburst risk and determining appropriate gas content threshold levels has identified significant deficiencies which are presented and discussed.

BACKGROUND

Following the fatal outburst that occurred at West Cliff colliery on 25 January 1994, the NSW Department of Mineral Resources issued a directive to operators of Bulli seam mines, which among the required actions intended to improve the management of outburst risk, specified gas content threshold limit values (TLV) for normal and outburst mining. As shown in Figure 1, the prescribed TLV for ‘normal’ mining was 9.0 m\textsuperscript{3}/t in 100% CH4 conditions and 5.0 m\textsuperscript{3}/t in 100% CO\textsubscript{2} conditions and the TLV for ‘outburst’ mining was 12.0 m\textsuperscript{3}/t in 100% CH4 conditions and 10.0 m\textsuperscript{3}/t in 100% CO\textsubscript{2} conditions.

Williams and Weissman (1995) presented a relationship between total measured gas content (QM) and a new term they referred to as Desorption Rate Index (DRI). The relationship between QM and DRI, which they had identified from analysis of gas emission data during gas content testing of core samples sourced from West Cliff colliery that contained high concentrations of CH4 and CO\textsubscript{2}, is presented in Figure 2. The data indicates a linear relationship between QM and DRI, represented by the equation, \( QM = \alpha \cdot DRI \), where the variable gradient of the trendline, \( \alpha \), representing the average of the data points, equals 0.01.
for >90% CH4 and 0.0067 for >90% CO2. This relationship between QM and DRI, identified from testing Bulli seam coal samples sourced from West Cliff colliery, was referred to as the Bulli Seam Benchmark. The relationship was assumed by Williams and Weissman (1995) to be representative of all Bulli seam conditions.

It was reported that the test to determine DRI involved measuring the volume of gas emitted from a 200 g sub-sample of coal material after crushing for 30 seconds and extrapolating the result to the total gas content (QM) of the full core sample to determine the DRI of the full coal sample (Williams, 1996 and Williams, 1997). The process to determine DRI, presented graphically by Williams (1997) is shown in Figure 3.

Using the Bulli Seam Benchmark, Williams and Weissman (1995) reported that gas content values of 9.0 m³/t for CH4 rich coal and 6.0 m³/t for CO2 rich coal, both corresponded to a common DRI value equal to 900 (Figure 4). They further proposed that, subject to determining the average gradient (α) of the QM-DRI relationship for a given coal seam, the QM value corresponding to a DRI of 900 represents the applicable outburst TLV for that coal seam. Figure 5 illustrates the method used to determine the equivalent outburst gas content TLV using the equation $Q_M = \alpha \times 900$, where $\alpha$ is determined for each coal seam. Mining in areas where the gas content level has been reduced below this threshold is intended to result in zero gas dynamic incidents and outbursts, regardless of the severity of any other condition (e.g. stress, degree of faulting, rate of mining).

Recent investigation and analysis of gas test data sourced from Australian coal seams, including the Bulli seam, has confirmed the two input variables required to calculate DRI are (a) the volume of gas released from a sub-sample of core in the initial 30 seconds after crushing during the Q3 phase of gas testing, adjusted to a standard sample mass of 200 g (Q3(30s)), and (b) the relative percentage of QM reported as Q3 (Q3/QM). As DRI is effectively a measure of the rate of gas release during mechanical crushing of coal it is
considered that it is incorrect and not appropriate to imply that DRI is a measure of desorption rate.

There has been increasing concern that DRI is overly simplistic and not a valid measure to fully assess and quantify outburst risk. Australia is the only country that uses a measure of gas emission from crushed coal as the basis for establishing outburst risk. China, Russia and other European countries employ measures of gas emission rate from fresh coal, during initial gas desorption. Examples of these measures of initial gas desorption used to assess outburst proneness include: $\Delta P$, $\Delta P_{0\text{ to }60}$, $\Delta P$ Express, $K_I$ Index, $V_{30}$ and $V$ Index, which effectively are measures of gas pressure or gas volume release from small sized particles collected in advance of working faces all of which aim to measure and compare gas emissions in the early stages of gas desorption from coal samples (Lama and Bodziony, 1996).

With reference to current Australian standards and practice, Initial Desorption Rate (IDR30) is a measure of the volume of gas desorbed from coal in the initial 30 minutes following collection of the core. IDR30 is routinely reported as part of gas content testing in accordance with Australian Standard AS3980:2016. Limited work has been undertaken to date to investigate the use of IDR30 as a potential indicator of outburst risk.

Another critical factor that impacts outburst risk is coal strength / coal toughness, and the ability of the coal to remain intact and avoid sudden brittle tensile failure due to mining induced fracturing and applied gas pressure. As the majority of outbursts that have occurred in Australian underground coalmines have been associated with geological structures, the impact and increased risk of outburst associated with structures must be considered.

The current ACARP funded research project (C26055) is investigating the relevance and applicability of the Bulli Seam Benchmark and DRI for use in assessing outburst risk and determining appropriate outburst threshold gas content values for Australian underground coalmines.

**BULLI SEAM BENCHMARK**

Raw gas emission data collected during gas testing on coal samples collected from areas of the Bulli seam rich in both CH4 and CO2 seam gas has been analysed to investigate the current Bulli Seam Benchmark and to determine whether the nature of gas emission from current mining areas has shifted from the original data presented in 1995 (Williams and Weismann, 1995). Figure 6 shows the relationship determined from the Bulli seam data collected to date. The results show a shift in the QM-DRI relationship for both CH4 and CO2, compared to the 1995 relationship, with increased DRI values relative to gas content.

Using records of gas emission during Q3 testing, the DRI calculation was repeated using the gas volume liberated from coal samples after the initial 60 seconds of crushing during Q3 testing, DRI(60s). The QM-DRI(60s) relationship for CH4 and CO2, presented in Figure 7, highlights the effect of increased gas volume release in the initial 60 seconds compared to the initial 30 seconds of Q3 crushing. Considering the extreme case of 100% of recorded Q3 gas emission being released from the coal samples within the initial 30 seconds of Q3 crushing, for both CH4 and CO2 rich coal samples, the resulting QM-DRI relationship is shown in Figure 8. This analysis confirms the significant impact and sensitivity that the measurement of gas volume released from coal samples during crushing has on the DRI.
Figure 6: Results of 2017 analysis of Bulli Seam Benchmark relationship for CH4 and CO2 rich coal samples.

The investigation of gas data collected during 2017 indicated a change in Bulli Seam Benchmark relationships for both CH4 and CO2 rich coal samples, presented in Figure 6, compared to the 1995 relationship presented in Figure 2. Given the impact that crushing efficiency and rate of gas release during Q3 has on DRI, a direct comparison of Q3 gas emission from two similar coal samples tested at different laboratories confirmed a difference in crushing efficiency and rate of gas release, shown in Figure 9. Further investigation also confirmed differences in crushing equipment used at two separate gas test laboratories. As shown in Figure 10 and Figure 11 respectively, Lab 1 uses twin puck and Lab 2 uses a single puck arrangement in the bowls of their ring mill crushers.

Analysis of the recorded gas emission from CH4 and CO2 rich coal subsamples weighing approximately 200 grams, after crushing in the Lab 1 ring mill for 30 seconds, confirmed initial gas flow rate from CO2 rich coal was consistently faster than from CH4 rich coal. Figure 12 and Figure 13 show the impact of seam gas composition on the percentage of Q3 gas content released from crushed coal samples in the initial 30 seconds (Q3(30s)/Q3) and the initial 60 seconds (Q3(60s)/Q3) of flow measurement. For CH4 rich coal samples, QM ranging from 4.0 to 11.0 m$^3$/t, Q3(30s)/Q3(Total) varied between 64 to 74% whereas for CO2 rich coal samples, QM ranging between 2.0 and 17.0 m$^3$/t, except for one sample, Q3(30s)/Q3(Total) varied between 83 to 95%.
Figure 9: Laboratory comparison of gas release rate during Q3 crushing indicating difference in crushing efficiency.

Figure 10: Lab 1 – twin puck ring mill Q3 coal crusher bowl.

Figure 11: Lab 2 – single puck ring mill Q3 coal crusher bowl.

Figure 12: Percentage of Q3 gas release in first 30 seconds of crushing CH4 and CO2 rich coal (Q3(30s)/Q3) presented relative to sample QM.

Figure 13: Percentage of Q3 gas release in first 60 seconds of crushing CH4 and CO2 rich coal (Q3(60s)/Q3) presented relative to sample QM.
Further investigation of gas emission data from CH4 and CO2 rich coal samples that impact the DRI calculation (a) volume of gas released from crushed coal in the initial 30 seconds of Q3 gas content testing (Q3(30s)), and (b) percentage of total gas content recorded as Q3 (Q3/QM), are presented in Figure 14 and Figure 15. The data shows that, for a given gas content (QM), (a) the volume of gas released in the initial 30 seconds of Q3 crushing from CH4 rich coal is less than from CO2 rich coal, and (b) the volume of gas measured during Q3 (Q3/QM) tends to be greater in CH4 rich coal, indicating a larger component of QM is released during the Q1 and Q2 stages of gas content testing from CO2 rich coal.

Figure 14: Gas volume released in initial 30 second of Q3 crushing from CH4 and CO2 rich coal samples relative to QM.

Figure 15: Percentage QM recorded as Q3 during gas testing CH4 and CO2 rich coal samples relative to QM.

DESORPTION RATE INDEX – DRI

Using the DRI approach to assess outburst proneness was regarded by Williams (2002) as being directly related to the desorption rate of the coal. However, the investigation of Bulli Seam Benchmark has shown that DRI is extremely sensitive to small changes in gas testing procedures, in particular (a) the time when the Q2 phase of gas testing is concluded, (b) time to break core and prepare subsamples of core material for use in Q3 testing, and (c) the equipment and energy applied to crush the coal during Q3 testing. Other potential limitations in the use of DRI to assess outburst risk and the use of DRI900 to determine outburst threshold levels has been investigated.

Figure 16 presents results from gas content testing on coal core samples collected from a CH4 rich non-Bulli seam mine, which includes the reported gas content component values, Q1, Q2 and Q3, IDR30 and DRI for core samples ranging in gas content from 3.0 to 14.1 m³/t. The graph shows DRI closely aligns with QM, whereas variability in Q1 and IDR30 does not have any impact on DRI.

Figure 16: Sample of reported gas test data and DRI sourced from a CH4 rich, non-Bulli seam mine (M9)
Figure 17 presents a comparative analysis of 21 core samples with gas content (QM) = 10 m⁴/t, to show the impact that (a) change in the relative percentage of QM reported as Q1, Q2 and Q3, and (b) volume of gas recorded at Q3(30s), has on the calculated DRI value for each coal sample. Comparing the results of samples 3 and 21, both samples having QM = 10.0 m⁴/t and DRI = 1700; the Q3 of sample 3 is 4.0 m³/t (Q3/QM = 40%) and Q3(30) = 510 mL, and the Q3 of sample 21 is 7.0 m³/t (Q3/QM = 70%) and Q3(30) = 893 mL. This example highlights how two coal samples with significantly different gas emissions characteristics can produce equal DRI values.

Figure 18 compares reported gas test results from six (6) Australian coal mines, each sample having QM = 10 m⁴/t and DRI = 1200. The results highlight the variability that can occur in the reported gas content component values, IDR30 and Q3(30s), without impacting the DRI value.

Lama and Bodziony (1996) discuss a number of outburst prediction indices that have been used in different countries. Whilst each index may vary in some way, whether it be (a) sample particle size or sample mass, (b) measured volume or pressure of desorbed gas, or (c) duration of measurement period, all methods test fresh coal and focus on initial desorption rate. In the current Australian Standard for gas content testing, AS3980:2016 (SAA, 2016), the only measure of initial gas desorption rate is the IDR30 which is a measure of the volume of gas released from a coal samples in the initial 30 minutes immediately following sample collection, measured in m³/t. The relationship between DRI and IDR30 relative to gas content (QM) for both CH₄ and CO₂ rich coal samples has been considered. Gas data from testing CO₂ rich coal sourced from reference mines M1 and M15 is presented in Figure 19, and gas data from testing CH₄ rich coal from reference mines M8 and M12 is presented in Figure 20. The gas data from the mines presented in both figures shows that while the average relationship between DRI and QM is linear, there is a non-linear and notable increase in IDR30 from samples with higher QM.
OTHER FACTORS THAT IMPACT OUTBURST THRESHOLD LIMITS

Many theories have been presented regarding the type and significance of factors that contribute to the occurrence of coal and gas outbursts (Black et al., 2009). Lama (1995) listed the following five factors considered to have the potential to contribute to an outburst:

- Tensile strength of coal;
- Gas emission rate;
- Gas pressure gradient;
- Moisture level; and
- Depth or stress level

Lama (1995) reported that from previous studies, gas was considered the major contributing factor to outburst occurrence in the Bulli seam. Gas content has therefore used as the primary indicator of outburst risk in all Australian underground coal mines and, where gas content is found to be at levels above the threshold limit, gas drainage is used to reduce gas content below the threshold level prior to mining (Black and Aziz, 2008). There are however many other factors that are relevant, and should be considered, in an assessment of outburst risk (Lama and Bodziony, 1996, Black et al., 2009, Gray et al., 2016). The factors considered to have the most significant impact on outburst risk have been presented in the outburst risk matrix shown in Figure 21.
Black and Aziz (2010) reported several Bulli seam mines that had introduced increased outburst threshold levels and discussed concerns that the Bulli Seam Benchmark and use of DRI900 as the basis for determining outburst threshold limits may not be appropriate.

Recent investigations into Australian outburst history and mining experience in areas where gas content was above the 1994 Bulli seam outburst threshold limits has provided no evidence that outburst had occurred at gas content levels below approximately 9.5 m$^3$/t, independent of gas composition. Most outburst events are associated with abnormal geological conditions. Walsh (1999) reported that of the approximately 250 outbursts that had been recorded at West Cliff colliery at that time, 70% occurred on strike-slip faults, 4% on dykes and faults, 1% on thrusts, 3% on normal faults; and 19% on bedding slips.

Figure 22 provides a summary of core sample gas test results from areas where gas content was above the ‘normal mining’ outburst threshold limit, that were mined using non-standard mining methods. Subject to the mine and their respective outburst risk management process, non-standard mining methods may include fully remote mining, grunching (shotfiring) and mining at reduced advance rate (limited rate mining). Tahmoor colliery has utilised limited rate mining for more than 15 years without an outburst, through both structured and non-structured coal, with gas content up to 12.0 m$^3$/t (CH4) and 10.0 m$^3$/t (CO2) (Borg, 2014).

The experience at Tahmoor colliery demonstrates the ability to successfully manage outburst risk, to enable mining to be carried out, without outburst, in areas where gas content is greater than the 1994 threshold limit for ‘normal mining’.

Further work is required to determine safe threshold limits, considering the key factors that impact outburst risk. Research is continuing, in conjunction with the University of Wollongong, to develop an Outburst Risk Index that considers other factors in addition to gas content/pressure, such as coal toughness, that may be used to assess outburst risk in Australian underground mines.

CONCLUSIONS

Investigations into the characteristics of the Bulli Seam Benchmark, using gas data collected from areas recently mined in the Bulli seam, has identified changes in the average QM-DRI relationship compared to data presented in 1995. Further investigation of the method used to calculate DRI has highlighted that (a) the performance of the crushing equipment, and (b) the
crushing and gas emission measurement procedure used to determine Q3, has a significant impact on the DRI value, which also affects the Bulli Seam Benchmark.

Investigations into DRI and the factors that affect the QM-DRI relationship demonstrated that the average QM-DRI relationship for each coal seam varies in accordance with the relative percentage of total gas emission recorded during Q3 testing that is released in the initial 30 second of crushing, i.e. Q3(30s)/Q3(Total). Moreover, the use of DRI incorrectly assumes that the rate of gas release from a combined mass of 150 or 200 grams of mechanically crushed coal, during Q3 residual gas content testing, is a measure of gas desorption rate. DRI is the only measure used to assess outburst risk and define outburst thresholds limits that is based on measurement of the gas emission rate from crushed coal in the later stages of gas content testing.

Gas content is considered to have the most significant impact on outburst risk and gas drainage to reduce gas content to safe levels plays a significant role in control and reduction of outburst risk in Australian underground coal mines. There are other significant factors that affect outburst risk and mining experience has demonstrated that where outburst risk factors, such as abnormal geological conditions are not present, that mining can be conducted without outburst at gas content levels greater than current normal mining threshold limits, and greater than those presently determined using the DRI900 method.

Further work will continue in association with the University of Wollongong to (a) investigate and determine threshold limits appropriate for other outburst risk indicators, such as coal toughness and gas pressure, and (b) develop a multi-factor Outburst Risk Index appropriate for assessing outburst risk in Australian mining conditions.

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REFERENCES


CONTROL AND MANAGEMENT OF OUTBURST IN AUSTRALIAN UNDERGROUND COAL MINES

Dennis J. Black¹,²

ABSTRACT: Outbursts represent a major safety hazard to mine personnel working near the coal face in areas of increased outburst risk. There have been over 878 outburst events recorded in twenty two Australian underground coal mines and most outburst have been associated with abnormal geological conditions.

Details of Australian outburst incidents and mining experience in conditions where gas content was above current threshold levels is presented and discussed. Mining experience suggests that for gas content below 9.0 m³/t, mining in CO₂ rich seam gas conditions does not pose a greater risk of outburst than mining in CH₄ rich seam gas conditions.

Mining experience also suggests that where no abnormal geological structures are present, mining in areas with gas content greater than the current accepted threshold levels can be undertaken with no discernible increase in outburst risk.

The current approach to determining gas content threshold limits in Australian mines has been effective in preventing injury from outburst however operational experience suggests the current method is overly conservative and in some cases the threshold limits are low to the point that they provide no significant reduction in outburst risk.

Other factors that affect outburst risk, such as gas pressure, coal toughness and stress and geological structures are presently not incorporated into outburst threshold limits adopted in Australian mines. These factors and the development of an Outburst Risk Index applicable to Australian underground coal mining conditions is the subject of ongoing research.

INTRODUCTION

Outburst has been defined as the sudden release of gas and material from the working place that can vary in magnitude and intensity (NSWDMR, 1995). The occurrence of an outburst is preceded by failure of the coal and during an outburst, the failed material is ejected with energy and with gas. The difference between a rockburst and an outburst is the gas that is emitted. The gas contributes in a major way to the expulsion of the coal and is generally thought to be the main contributor to total energy release (Gray and Wood, 2013).

Outbursts of coal and gas have been experienced in underground coal mines in many countries, including Australia (Lama and Bodziony, 1996). Outburst vary in size and intensity, from small bumps equivalent to rib failure without discernible gas release to violent ejections of thousands of tonnes of coal and rock releasing tens of thousands of cubic metres of seam gas. The sudden release of a large volume of seam gas into a mining place following an outburst can create significant potential risks to personnel safety which include: (a) danger of asphyxiation due to oxygen deficiency, (b) poisoning by noxious gases, (c) explosion by inadvertent ignition of the resultant explosive mixtures, (d) injury resulting from the violent ejection of coal and gas, and (e) exposure to dense coal dust.

Gas, geology and stress have been identified as dominant parameters that combine to create outburst conditions and provide the energy required to expel coal from the working face (Black et al., 2009). Outbursts are usually, but not invariably associated with faults, dykes, seam variations and dislocations. In some mines, such as Leichhardt colliery in Queensland some outbursts occurred in areas with no abnormal geological structure or with structures which elsewhere in the same mine had been quite benign (Hanes, 1995).

The general nature of the outburst risk is such that it may be continuously variable, not only between mines but also within an individual colliery’s workings. A single, unchanging approach to the management of the risk is, therefore, inappropriate. A degree of discipline is also warranted to identify, and effectively act upon, changes in the mine operating conditions.

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environment, which may be subtle, which may be associated with the potential for outbursts (NSWDMR, 1995).

**AUSTRALIAN OUTBURST HISTORY**

The first recorded outburst in Australia occurred at Metropolitan colliery on 30 September 1895 and in the 122 year period to 2017, over 878 outburst events have been recorded in twenty two Australian underground coal mines. Mining operations in the Bulli seam, located in the southern Sydney basin, have the longest history of outburst in Australia, with fourteen collieries recording over 641 outburst incidents. Ellalong colliery, working the Greta seam in the northern Sydney basin, recorded five relatively small-scale outburst events, and is the only non-Bulli seam mine in New South Wales to record an outburst event. Seven collieries operating in the Bowen basin, Queensland, have recorded over 232 outburst incidents, with the largest number of events recorded during development mining in the Gemini seam at Leichhardt colliery. Table 1 provides a summary list of recorded outburst data compiled from extensive review of published reports and Mines Department records.

The lives of twenty-one men and five horses have been lost in eight outburst incidents in Australian underground coal mines. The largest outburst event recorded in Australia, which claimed the lives of seven men and three horses, occurred at the Collinsville State mine in Queensland on 13 October 1954. In three separate outburst incidents, seven men and two horses were killed at Metropolitan colliery. Table 2 provides a summary list of fatal outburst incidents that have occurred in Australian underground coal mines.

Table 1: Summary of Recorded Outburst Incidents in Australia

<table>
<thead>
<tr>
<th>Mine</th>
<th>State</th>
<th>Seam</th>
<th>Recorded Outburst Date Range</th>
<th>Recorded Outburst Event Code Range</th>
<th>Max. Coal Outburst (tonnes)</th>
<th>Max. Gas Outburst (m³)</th>
<th>Gas Type CH₄, CO₂</th>
<th>Associated Geocorrelation Structure</th>
<th>Gas Drainage Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apple</td>
<td>NSW</td>
<td>Bull</td>
<td>67 May 1999 - Feb 2017</td>
<td>&lt;40</td>
<td>5,160</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
</tr>
<tr>
<td>Barlomew</td>
<td>NSW</td>
<td>Bull</td>
<td>2 1992</td>
<td>&lt;40</td>
<td>16</td>
<td>Unidentified</td>
<td>Fracture</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Coal Cliff</td>
<td>NSW</td>
<td>Bull</td>
<td>2 November 1953</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Central (South Ball)</td>
<td>NSW</td>
<td>Bull</td>
<td>2 October 1953</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Dawes Foremost (closed)</td>
<td>NSW</td>
<td>Bull</td>
<td>2 January 1953</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Ellalong</td>
<td>NSW</td>
<td>Greta</td>
<td>5 October 1953</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Kanuma (closed)</td>
<td>NSW</td>
<td>Bull</td>
<td>2 May 1953 to May 1961</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Metropolitan</td>
<td>NSW</td>
<td>Bull</td>
<td>1695 Sep 1895 - Jan 2017</td>
<td>250</td>
<td>11,500</td>
<td>Artificial CH₄, CO₂</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
</tr>
<tr>
<td>North Bull (closed)</td>
<td>NSW</td>
<td>Bull</td>
<td>1 1991</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
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<td>Oakdale (closed)</td>
<td>NSW</td>
<td>Bull</td>
<td>1 1991</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
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<tr>
<td>Bull (closed)</td>
<td>NSW</td>
<td>Bull</td>
<td>3 1992</td>
<td>&lt;40</td>
<td>16</td>
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<td>Strike, Fracture, Normal Faulting</td>
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<td>No Drainage</td>
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<td>Bull</td>
<td>7 Feb 1995 to Apr 1996</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄, CO₂</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
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<tr>
<td>Tannenroder</td>
<td>NSW</td>
<td>Bull</td>
<td>90 May 1951 to Mar 1952</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
</tr>
<tr>
<td>Tower (Appin)</td>
<td>NSW</td>
<td>Bull</td>
<td>21 Jul 1991 to Dec 1996</td>
<td>80</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
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<td>West Cliff (Appin)</td>
<td>NSW</td>
<td>Bull</td>
<td>264 Dec 1976 to Apr 1996</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄, CO₂</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No or Inadequate Drainage</td>
</tr>
<tr>
<td>Central (closed)</td>
<td>QLD</td>
<td>Ceman Creek</td>
<td>1 20 Jul 2001</td>
<td>&lt;40</td>
<td>1,500</td>
<td>Artificial CH₄, CO₂</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>Inadequate Drainage</td>
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<tr>
<td>Collieville (closed)</td>
<td>QLD</td>
<td>Bowen</td>
<td>13 Mar 1980 to Mar 1991</td>
<td>300</td>
<td>14,000</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
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<tr>
<td>Collieville No 3 (closed)</td>
<td>QLD</td>
<td>Bowen</td>
<td>2 Mar 1977 to Apr 1972</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
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<td>No Drainage</td>
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<tr>
<td>Collieville No 2 (closed)</td>
<td>QLD</td>
<td>Bowen</td>
<td>7 Sep 1978 to Nov 1979</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
</tr>
<tr>
<td>Leichhardt (closed)</td>
<td>QLD</td>
<td>Grena</td>
<td>2044 1975 to 1992</td>
<td>&lt;40</td>
<td>16</td>
<td>Artificial CH₄</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>No Drainage</td>
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<tr>
<td>Moura No 4 (closed)</td>
<td>QLD</td>
<td>C-emer</td>
<td>3 1980 to 1983</td>
<td>&lt;40</td>
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<tr>
<td>North Gwyngells</td>
<td>QLD</td>
<td>Gwyngle Middles</td>
<td>8 20 Nov 2001</td>
<td>&lt;40</td>
<td>16,000</td>
<td>Artificial CH₄, CO₂</td>
<td>Strike, Fracture, Normal Faulting</td>
<td>Dyke</td>
<td>Inadequate Drainage</td>
</tr>
</tbody>
</table>

University of Wollongong, February 2018

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Table 2: Summary of Fatal Outburst Incidents in Australia

<table>
<thead>
<tr>
<th>Mine</th>
<th>State</th>
<th>Seam</th>
<th>Date</th>
<th>Loss of Life</th>
<th>Outburst Size (tonnes)</th>
<th>Gas Volume Gas Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metropolitan</td>
<td>NSW</td>
<td>Bulli</td>
<td>10 Jun 1896</td>
<td>3 men, 1 horse</td>
<td>Unknown</td>
<td>CH4</td>
</tr>
<tr>
<td>Metropolitan</td>
<td>NSW</td>
<td>Bulli</td>
<td>27 Jun 1925</td>
<td>2 men, 1 horse</td>
<td>220</td>
<td>Mixed CO2 &amp; CH4</td>
</tr>
<tr>
<td>Collieville State</td>
<td>QLD</td>
<td>Bowen</td>
<td>13 Oct 1954</td>
<td>7 men, 3 horses</td>
<td>500</td>
<td>14,000m³ CO2</td>
</tr>
<tr>
<td>Metropolitan</td>
<td>NSW</td>
<td>Bulli</td>
<td>02 Dec 1954</td>
<td>2 men</td>
<td>90</td>
<td>CO2</td>
</tr>
<tr>
<td>Leichhardt</td>
<td>QLD</td>
<td>Gemini</td>
<td>01 Dec 1978</td>
<td>2 men</td>
<td>350</td>
<td>13,500m³ CH4 + CO2</td>
</tr>
<tr>
<td>Tahmoor</td>
<td>NSW</td>
<td>Bulli</td>
<td>24 Jun 1985</td>
<td>1 man</td>
<td>400</td>
<td>4,500m³ CO2 + CH4</td>
</tr>
<tr>
<td>South Bulli</td>
<td>NSW</td>
<td>Bulli</td>
<td>24 Jul 1991</td>
<td>3 men</td>
<td>300</td>
<td>6,000m³ CO2 + CH4</td>
</tr>
<tr>
<td>West Cliff</td>
<td>NSW</td>
<td>Bulli</td>
<td>25 Jan 1994</td>
<td>1 man</td>
<td>350</td>
<td>CO2</td>
</tr>
</tbody>
</table>

Virtually all outburst events in the Bulli seam have been associated with geological structures and have occurred in areas where no substantial gas drainage has been undertaken (Lama, 1995). The highest number of outburst events have occurred at West Cliff, Metropolitan, Appin and Tahmoor collieries which all work the Bulli seam and Leichhardt colliery which worked the Gemini seam. Brief summaries of the outburst histories at these five collieries are provided.

**West Cliff Colliery**

West Cliff colliery commenced coal production in October 1976 and longwall mining was introduced in 1982 (Eade, 2002). The first recorded outburst occurred at West Cliff on 20 December 1976 and two hundred and fifty four (254) outburst events were recorded at West Cliff prior to the mine ceasing production in 2015-2016. The size of outbursts varied from four to over 300 tonnes, with the majority being related to zones of strike-slip faulting. Outbursts at West Cliff typically occurred in association with pulverised coal in shear zones running through the coal. The shear zones were also regions of high gas pressure, which when intersected, resulted in displacement of pulverised coal into the excavations (Marshall et al., 1980). Walsh (1999) reported that of the approximately 250 outbursts recorded at West Cliff, 70% occurred on strike-slip faults, 4% on dykes and faults, 1% on thrusts, 3% on normal faults; and 19% on bedding slips.

The largest outburst at West Cliff, reported to have displaced 320 tonnes of coal, occurred at the northwest end of a normal fault where the gas drainage holes had not penetrated. There was a major joint zone 3-4 m wide in the roof associated with this outburst site and a mylonite band some 30-mm thick. The gas composition had been predominantly methane. In the north-eastern part of the mine, outburst events had occurred in areas with high gas content (>16 m³/t) and high concentrations of carbon dioxide (>95% CO2) (Harvey, 1994).

Marshall et al. (1980) reported the time taken for an outburst to manifest at West Cliff varied from several seconds to almost a minute. In the opinion of Harvey (1994), mining operations at West Cliff had been possible through the use of gas drainage and specified outburst mining procedures.

On 03 April 1998, West Cliff became the first mine in Australia to record an outburst on a retreating longwall face. Two outbursts of approximately 17 tonnes were identified on the longwall face (LW23) at Chocks #45 and #54 during the flit run of the cutting cycle (Walsh, 1999). The outbursts, comprising some fines but mostly blocky coal (Piper, 1998) were identified as cones extending into the face (>1.0 metre) at the top of the 2.5 metre Bulli seam. Two hours after the outbursts, gas continued to be liberated from the cavities and could be heard as an audible hiss and a visible haze (Walsh, 1999). No abnormal structures, such as strike-slip or thrust faults were noted at the outburst sites, however a bedding plane slip was present in the seam, approximately 100 mm below the roof. There was also no prominent cleat noted at the outburst sites (Walsh 1999).
Metropolitan Colliery

Mining operations at Metropolitan colliery commenced in 1888 with the Bulli seam being mined by hand, with some single round shotfiring (Ward, 1980). Metropolitan was the first colliery in Australia to record an outburst, which occurred on 30 September 1895. The mine has since recorded over 169 outburst events.

The highest incidence of outburst occurred during the mining of the 2 South District between 1961 and 1968, where over 100 outburst events were reported to have been induced by shotfiring. The largest reported outburst ejected 250 tonnes of coal and an unknown quantity of predominantly CO2 (Lama, 1995).

On 23 December 2016, Metropolitan became the second mine in Australia to record outburst events on a retreating longwall face. A total of three outburst / slump events occurred in quick succession as the longwall (LW27) retreated through a significant thrust fault zone. The largest of three outburst events occurred on 04 January 2017, ejecting approximately 200 tonnes of coal and releasing approximately 11,500 m$^3$ of CO2 (Hyslop, 2017).

Gas content at Metropolitan has been recorded above 20 m$^3$/t and the seam gas composition in the current mining area is predominantly CO2. Early outbursts were recorded as fire damp (CH4) and recent outbursts are of black damp (CO2) (Chatterjee, 1982).

A review of relevant reports and information indicates that the majority of the outbursts occurred on structures, especially a zone known at the mine as the “soft outburst zone” (Harvey, 2002).

Appin Colliery

Appin commenced operation in 1962 and longwall mining was introduced in 1969 (Eade, 2002). The first recorded outburst occurred in May 1966, ejecting 50 tonnes of coal and an unknown quantity of CH4. The outburst occurred in a zone of joints that were evident in the immediate roof (Harvey, 2002). A total of 67 outburst have been recorded at Appin. Twenty outbursts events were recorded in the 27 years to 1994, the largest occurred in July 1969 when development mining intersected a strike-slip fault with mylonite displacing 100 tonnes of coal and an unknown volume of CH4 (Lama, 1991).

In the years following 1994, 47 seven outbursts have been recorded at Appin, the largest reported in May 2009 while operating a remote controlled continuous miner to develop through a known thrust fault zone that had been difficult to drill and drain gas below the outburst threshold. The outburst displaced 150 tonnes of coal and released 1,140 m$^3$ of predominantly CH4.

The largest reported outburst to have been induced while shotfiring through a dyke associated with strike-slip faulting occurred in January 2013 and released an estimated 5,100 m$^3$ of predominantly methane. The mass of coal displaced by the outburst, in addition to the planned shotfiring excavation, was not reported.

Outburst events at Appin typically occur in areas where prominent geological features have been intersected. Such features include faults, particularly strike-slip and thrust faults, adjacent to dykes and associated cindered coal. Harvey (2002) reported five small outbursts, four being less than eight tonnes and one of up to 20 tonnes, had occurred in areas where ‘no prominent geological structure’ had been identified.

Gas content at Appin has been measured at levels exceeding 16 m$^3$/t and an extensive gas drainage system is used to prevent or minimise the risk of outbursts and manage gas liberated during mining. Composition of the gas is predominantly CH4, however high CO2 has been recorded adjacent to faults and dykes (Harvey, 2002).

Tahmoor Colliery

Mine development commenced at Tahmoor in 1978 and longwall mining was introduced in 1986 (Newman, 2005). Ninety-nine (99) outbursts have been recorded at Tahmoor colliery in the years following the first recorded outburst in 1981. The mine identified the developing outburst problem, with events progressing in significance from slumps and pressure bumps to large outbursts occurring on geological structures, particularly dykes and strike-slip faults (Stone, 1991, Newman, 2005 and Wynne and Case, 1995). Wynne and Case (1995) reported...
that all outburst events at Tahmoor had occurred during the cutting phase of the development mining cycle. Table 3 lists the structural association of outbursts recorded at Tahmoor (Stone, 1991).

Table 3: Summary geological structure association with outburst at Tahmoor colliery (Stone, 1991)

<table>
<thead>
<tr>
<th>Geological Structure</th>
<th>No. of Outbursts</th>
<th>Violent Outburst</th>
<th>Size of Outburst</th>
</tr>
</thead>
<tbody>
<tr>
<td>Across Dyke</td>
<td>3</td>
<td>3</td>
<td>5 - 400 t</td>
</tr>
<tr>
<td>Strike-Slip Fault / Dyke</td>
<td>27</td>
<td>17</td>
<td>5 - 120 t</td>
</tr>
<tr>
<td>Strike-Slip Fault</td>
<td>49</td>
<td>15</td>
<td>5 - 60 t</td>
</tr>
<tr>
<td>Reverse Fault</td>
<td>4</td>
<td>1</td>
<td>5 - 40 t</td>
</tr>
</tbody>
</table>

Following the fatal outburst incident that occurred in the 204 Panel on 24 June 1985, Tahmoor worked to modify the Joy 12 continuous miner to provide increased protection for the miner driver in outburst conditions. A completely enclosed cabin was built which was protected by one-inch thick bulletproof glass (Figure 1). Inside the cabin, the operator, who wore an air mask, communicated through radio control with the shuttle car driver and the crew at a fresh air base. The enclosed cabin was ventilated with two sources of fresh air supply, along with two additional sources of emergency air supply. Air pressure in the cabin was maintained to stop potential gas seepage into the miner’s cabin. The outburst miner was replaced in 1992 with the introduction of the remote controlled Alpine Bolter Miner (ABM20).

Figure 1: Joy 12 CM 20 machinery equipped with an outburst protection cabin, Tahmoor Colliery (Lama and Bodziony, 1998)

Wynne (2002) listed critical events and stages in the evolution of outburst management at Tahmoor, which include:

- 1981 – first recorded outburst;
- 1985 – continuous miner driver killed by outburst whilst cutting dyke;
- 1985 – encapsulated continuous miner introduced for cutting outburst structures;
- 1982 to 1992 – averaging 10 outbursts per year crossing structures;
- 1992 – introduced ABM20 continuous miner capable of being remotely operated;
- 1992 – commenced pre-drainage of coal around structures;
- 1992 – draft Outburst Management Plan;
- 1992 – remote mining through fault, last recorded outburst;
- 1992 to 1997 – ongoing refinement of drilling techniques;
- 1994 – Outburst Management Plan formalised;
- 1999 to 2001 – shotfiring (grunching) through “tight” coal zones.
Wynne (2000) reported there had not been an outburst event at Tahmoor Colliery since 1992, due to effective pre-drainage and the Outburst Management Plan. In 2003, Tahmoor introduced increased gas content threshold levels for outburst control, which included among other management controls, (a) increased drilling for pre-mining gas content reduction and compliance core sampling, and (b) limited rate mining.

**Leichhardt Colliery**

Leichhardt colliery was the most outburst prone underground coal mine in Australia, with more than two hundred outbursts reported between 1973 and 1978. Leichhardt colliery commenced operations in the Gemini seam in 1973 and, following the fatal outburst in December 1978, the mine was placed on ‘care and maintenance’ and operated as an experimental mine for four years before closing in December 1982.

The average thickness of the Gemini seam was 6.0 m and the mine operated at a depth of 350 to 410 m. Within the mine, shallow dipping reverse faults of minor displacement and associated slickensides were common. Seam gas was predominantly CH4 (95% CH4 / 5% CO2). Characteristics of the Gemini seam included high desorption rate, high gas pressures, low permeability and the gas content was around 15 m³/t (Truong et al., 1983 and Hanes, 2001).

Mining was done by continuous miners, shotfiring and an Alpine Roadheader. Drilling large diameter boreholes in advance of mining was used for a period to reduce stress and gas emissions and was considered to improve the mining conditions. The practice was later abandoned due to a belief that large diameter boreholes were no more effective than smaller diameter holes and operational problems were associated with drilling large diameter boreholes. Other preventive techniques such as shotfiring, and delayed action shotfiring were also practised which generally reduced the frequency of outburst occurrences (Truong et al., 1983).

Mining induced cleavage was typical in the coal at Leichhardt. It curved around the face forming large sheets of coal which easily spalled or at times, burst. Outbursts had not occurred in the western workings of the mine due to changes in the gas and/or structural regime and the coal which was free from bursts lacked the mining induced cleavage (Moore and Hanes, 1980 and Hanes, 2006). In the eastern part of the mine, outbursts occurred frequently (daily) from the rib (Moore and Hanes, 1980). The outbursts were typically small and occurred as the violent buckling of a few tonnes of the cleated coal into the opening. At times, the miner driver could “turn on” an outburst for visitors (Moore and Hanes, 1980).

Typically, outburst prone coal was intensely cleaved around the mine opening. Outbursts were partly controlled by stress and by the cleats. Drives near parallel to the maximum principal stress were free from outbursts and other mining strain, whereas drives nearly perpendicular to the principal stress were highly strained and outburst prone. The rib which first intersected the cleat was the focus of most bursts, which projected perpendicular to the cleat (Hanes, 2001). Outburst cavities in the Gemini Seam were typically oriented such that their axes were perpendicular to the face cleat direction and the bursts occurred from the ribs or face generally on the side which first encountered the cleat (Moore and Hanes, 1980). Some outbursts occurred with their axes perpendicular to prominent induced cleavage and many bursts occurred from coal roof with their axes perpendicular to the bedding planes (Hanes, 1979).

The orientation of the fatal outburst that occurred on 01 December 1978 was apparently controlled by cleat orientation. The axis of the burst cavity was approximately perpendicular to the dominant cleat direction over most of its length. Also, the axis of the burst cavity was nearly parallel to the mean plane of slickenside planes in the burst cavity walls (Hanes, 1979). Tight ribs preceded most outbursts. Pick marks were obvious for the full height of the seam. The face at the fatal 1978 outburst had very hard coal ribs which “rang” when hit with a hammer (Hanes, 2006).

Measured gas pressure gradients showed that gas pressure was the controller of outbursts. When the pressure gradient in the face was high, outbursts occurred (Hanes, 2001). Gas flow measurements showed that on drilling, gas did not flow from drainage holes until the holes had stood for 2 to 3 months (Hanes, 2001). Figure 2 shows the results of pressure
measurements in the coal ahead of the working face in outburst and benign conditions reported by Hanes (1995).

![Figure 2: Gas pressure and content gradients recorded in the Gemini seam, Leichhardt colliery (Hanes, 1995)](image)

**INTRODUCTION OF OUTBURST THRESHOLD LIMITS IN BULLI SEAM MINES**

The earliest attempts to develop safe threshold values for mining the Bulli seam were based upon measurement of gas emission rate from freshly cut coal (Lama, 1995). A version of the French, Belgian and Polish gas emission meter was introduced to Australian mines by Hargraves, where a 4.0 gram coal sample of -14 to +25 mesh fraction was collected and gas emission measured over a 2 to 6-minute period (Lama, 1995). Indices were developed which showed that if the gas emission was greater than 1.5 cc/g for CH4 and 1.2 cc/g for CO2, then the face was liable to outburst (Lama, 1995). The method required indices to be developed for each site, to suit local conditions, yet based on work in French mines the index value CO2 areas was dropped to 1.0 cc/g (Lama, 1995). There were several problems with this method which affected the accuracy and repeatability of the measurement, which included moisture, variability of coal ply and depth of drill hole from which the sample was sourced. These issues, combined with the introduction of high performance roadway development systems to support the introduction of longwall mining rendered the method, which required frequent gas emission measurement of coal samples collected from 2-3 metres holes drilled ahead of the advancing working faces, unsuitable as it adversely affected productivity and the results were considered unreliable (Lama, 1995).

Early attempts at drilling larger diameter boreholes, up to 300 mm diameter, ahead of the working face as a means of reducing stress also aided in draining gas and reducing gas emissions thereby reducing outburst risk (Lama, 1995). Lama (1995) reported gas drainage investigations at Tahmoor showed that when an area had been drained to gas levels between 9.0 - 10.7 m³/t, with CO2 percentage 40 - 45%, there were no violent outburst events, even when structures such as dykes were present in the area. Lama also reported an outburst with the emission of almost 3000 m³ of gas occurred in an area where gas content was between 11 - 12 m³/t, with gas pressure of 1700 kPa. When gas levels were dropped to 6.0 m³/t, no outbursts occurred. Work at Metropolitan colliery found the Bulli seam had been mined without outburst in areas where the gas emission value was below 0.6, with desorbable gas content of 4.0 m³/t in 90% CO2 (Lama, 1995).

It should be noted that gas content values reported by Lama were desorbable gas content (Q1 + Q2) as the gas content test method did not routinely measure the Q3 residual gas content component. The test method used by Lama to determine the desorbed gas content involved measuring gas desorbed from coal samples collected from coal samples over a maximum 48 hour period, or the time when at least one negative gas emission value was observed as a result of minor pressure and temperature changes causing resorption of gas from the surroundings into the coal sample (Lama, 1995).

Lama (1995) provided a brief description of work to measure the residual gas content of a selection of Bulli coal samples which, with some corrections, average Q3 values of 2.01 m³/t
for CH4 and 2.4 m³/t for CO2. The approach taken by Lama was to add the average residual gas content values (Q3) to the measured desorbed gas content (Q1 + Q2) values to report total gas content (Q1 + Q2 + Q3). Results of gas emissions from slow desorption testing of coal samples by Black (2011) raises concerns for the accuracy of Lama's approach to determining total gas content, as indicated by gas emission measurements of two coal samples, shown in Figure 3, which show gas desorption occurs for a substantially longer period than 48 hours with Q3 greater than 2.0 m³/t being measured from coal core after slow desorption testing for a period of 600 days.

Based on the results of gas content measurements and a review of gas content threshold values used by other countries, such as Poland, Russia, Germany, Bulgaria and China, Lama proposed gas content threshold values based on desorbable gas content (1991) and total gas content (1995), having added the average Q3 test results to the desorbable gas content threshold values. The gas content threshold values proposed by Lama, for both desorbable and total gas content, are presented in Table 4.

![Figure 3: Examples of gas emission rate from coal core during slow desorption testing (Black, 2011)](image)

Table 4: Lama’s recommended gas threshold values for safe mining of Bulli seam (total gas content) (Lama, 1991 and 1995)

<table>
<thead>
<tr>
<th>Proposed Outburst Threshold Limits [Seam Status]</th>
<th>Desorbable Gas Content (m³/t) (Lama, 1991)</th>
<th>Total Gas Content (m³/t) (Lama, 1995)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1 TLV: Presence of Structure</td>
<td>8.0 (100% CH4) 4.0 (100% CO2)</td>
<td>8.0 + 2.0 = 10.0 4.0 + 2.4 = 6.4</td>
</tr>
<tr>
<td>Level 2 TLV: No Structures Present</td>
<td>10.0 (100% CH4) 7.0 (100% CO2)</td>
<td>10.0 + 2.0 = 12.0 7.0 + 2.4 = 9.4</td>
</tr>
</tbody>
</table>

In 1992 the NSW Chief Inspector of Coal Mines (CICM), concerned about the increasing number of outburst incidents, indicated by data presented in Figure 4, reported in Annual Reports of the NSW Department of Mineral Resources, and the apparent lack of effective management of outburst risk, formed specialist work groups to identify the regional characteristics of outbursts and develop the most appropriate means of protecting mine workers (Harvey, 1994). The working group identified the need for management plans, and all mines operating in the Bulli seam were requested to prepare outburst management plans to specify how they would manage outburst risk (NSWDMR, 1992, Harvey, 1994). The objective of the NSW Department of Mineral Resources (DMR) was that all Southern Coalfield mines would be operating under auditable outburst management plans by 30 June 1993. However, the plans submitted to the DMR were generally regarded as inadequate and were returned to the mines for further development. During 1992 and 1993, the CICM was considering the introduction of gas content threshold limit values as a means of reducing outburst risk. During this process, gas content TLVs as low as 6.0 m³/t in 100% CH4 conditions and 3.0 m³/t in 100% CO2 were considered.
Following the fatal outburst at West Cliff colliery on 25th January 1994, the DMR issued a notice to all mines operating in the Bulli seam pursuant to Section 63 of the Coal Mines Regulation Act 1982, detailing actions to be implemented to prevent further outburst related fatalities. Arguably the most significant of these actions was the stipulation of limits on seam gas content prior to mining, known as outburst threshold limit values (TLV). Figure 5 shows the Bulli seam TLV prescribed in the Section 63 notification (Clarke, 1994). The TLV varied linearly based on gas composition, with the presence of CO2 seam gas considered a significantly higher outburst risk than CH4 seam gas. The Level 1 TLV for ‘normal’ mining was 9.0 m$^3$/t in 100% CH4 conditions and 5.0 m$^3$/t in 100% CO2 conditions. If gas content was not reduced below the Level 1 TLV, mining was only permissible under outburst mining procedures. The Level 2 TLV for ‘outburst’ mining was 12.0 m$^3$/t in 100% CH4 conditions and 10.0 m$^3$/t in 100% CO2 conditions. If gas content was not reduced below the Level 2 TLV, mining was only permissible using remotely operated equipment, with all personnel remaining clear of the outburst risk zone.

The introduction of TLV resulted in a significant increase in the intensity of drilling and gas drainage to identify geological structures and reduce gas content below threshold limits. Mine operators developed comprehensive outburst management plans which included standard drilling patterns and routine management controls to deal with the issue of gas content reduction. However, these TLVs preceded the introduction of intensive inseam gas drainage and the capability of directional drilling technology to aid in locating geological structures and other outburst risk zones.

Lama (1995) presented gas content data collected over a three (3) year period from sites where headings had been mined through structures with and without outbursts. With reference to the gas content and outburst data, presented in Figure 6, Lama proposed the
two threshold limits lines; solid TLV line for areas with structures present and dotted TLV line for areas without structures. Lama stated the proposed values were safe and the safety factor of 19% (1.1 m³/t) was greater than the error associated with gas content measurement. Points lying between the two TLV lines show small outbursts occurring on structures. Details of the size of the outburst events, stated by Lama to be “too small to cause any major damage or endanger life”, are also presented in Figure 6. The gas content TLVs proposed by Lama for safe mining of the Bulli seam, based on the presence or absence of geological structures, and based on the work presented in Figure 6, have been summarised in Figure 7. The proposed Level 1 TLV of 6.4 m³/t for CO₂ and 9.4 m³/t for CH₄ was considered safe under all circumstances, i.e. when mining near geological structures with a development advance rate up to 50 m/d. Lama also suggested that if the rate of development advance was reduced to 10-12 m/d, the Level 1 TLV could be safely increased by 20%. The Level 2 TLV of 10.0 m³/t for CO₂ and 12.0 m³/t for CH₄ was proposed for development mining in areas where no geological structures were present within 5.0 m of the excavation.

Details of the outburst incidents referred to by Lama (Figure 6) have not yet been located for review and verification of prevailing conditions. While details of the timing and proximity of the gas content sample locations relative to the outburst reference points A to I were not reported by Lama, the description of “Material thrown out = nil” suggests the incidents may have been gas blowers rather than outbursts. Also, given the gas content testing method used by Lama involved measurement of desorbed gas content and the addition of average residual gas content (Q3) values to determine ‘total gas content’, it is suggested potential error may be present in the reported gas content values.

Figure 6: Recorded Total gas content data close to structures, Tahmoor and West Cliff mines (Lama, 1995)

Figure 7: Lama’s recommended Bulli seam Outburst Threshold Limits (Lama, 1995)
The gas content TLV prescribed by the DMR (Figure 5), which are lower than the TLVs proposed by Lama (Table 4 and Figure 7), indicates an additional level of conservatism and increased ‘factor of safety’ was applied by the DMR. Although the prescribed TLV was conservative (Black et al., 2009), the lower gas content threshold did achieve the objective of the DMR, which was to eliminate fatal outburst incidents in the Bulli seam. The removal of gas by gas drainage and the reduction of gas content to safe levels were uncritically accepted by the mining industry (Lama, 1995). Favourable conditions present in the mines at that time enabled the seam gas content to be reduced below TLV relatively easily and without delay to mine operations and coal production.

Impact of seam gas composition on outburst risk

The outburst TLVs adopted in Australian mines reflect the view that mining in coal with high concentrations of CO2 represents a significantly greater risk of outburst in comparison to mining in coal with high concentration of CH4. Commonly reported views of researchers suggest (a) CO2 is more outburst prone than CH4 (Lama, 1995 and Williams, 2000), (b) CO2 outbursts are more violent than CH4 outbursts (Hargraves, 1980, Hanes, 2001, Lama and Saghafi, 2002), and (c) CO2 reduces the strength of coal (Wu, et al., 2010). Wynne (2002) questioned the difference in threshold limit for CO2 and CH4, posing the question “is an outburst more likely in a CO2-rich seam than in a CH4-rich seam?”.

Results of gas content and gas composition data collected from core samples near recorded outburst events in Australian underground coal seams, presented in Figure 8, highlight the absence of outburst events below approximately 9.0 m3/t, specifically in conditions where CO2 is the dominant seam gas.

![Figure 8: Gas content and composition measurement near recorded Australian outburst events](image)

The basis for the commonly held view that outbursts associated with carbon dioxide are more violent, more difficult to control and more dangerous appears to be due to the greater sorption capacity of coal for carbon dioxide (Hargraves, 1980, Hanes, 2001, Lama and Saghafi, 2002). Laboratory based experimentation of outburst propensity using briquettes formed from pulverised coal in small-scale outburst simulation apparatus, such as those described by Skoczylas (2012), Wang et al. (2015) and Zhao et al. (2016), do not discuss the fact that when comparing the burst response of coal samples saturated with CO2 and CH4 at the same pressure, the effective gas content of the CO2 test sample will be approximately twice that of the CH4 test sample, due to the inherent sorption characteristics of coal. Consequently, for a given coal seam gas pressure, coal samples will contain a larger volumes of carbon dioxide and emission problems therefore appear more acute (Beamish and O’Donnell, 1992).

The sorption capacity of Bulli seam coal for both CO2 and CH4, as reported by Black (2011) and presented in Figure 9, highlight the increased sorption capacity of CO2 in comparison to
CH4. With reference to the isotherms for CO2 and CH4 presented in Figure 9, and considering the measurement of gas content is the principal measure of outburst risk; for a given gas content value the isotherms indicate the gas pressure of the CH4 rich sample will be substantially greater than the CO2 rich sample and therefore CH4 rich coal potentially contains gas at higher pressure than the CO2 rich coal and therefore potentially represents a greater outburst risk.

Figure 9: Example Bulli seam isotherm curves for methane and carbon dioxide sorption (after Black, 2011)

With respect to the impact of CO2 on coal strength, the effects of sorption induced swelling and potential weakening of coal samples observed in laboratory testing does not typically relate to in situ conditions experienced in Australian coal seams containing CO2. Australian coal seams containing high concentrations of CO2, such as the Hoskissons seam mined at Narrabri, the Greta seam mined at Austar and the Bulli seam mined at Metropolitan, Tahmoor and Appin do not routinely experience weakened coal conditions.

OUTBURST THRESHOLD LIMITS APPLICABLE TO NON-BULLI SEAM MINES

Outburst thresholds in non-Bulli seam mines are established based on the GeoGAS Desorption Rate Index (DRI) and acceptance of the outburst mechanism where the desorption rate of gas is directly used as an indicator of outburst proneness (GeoGAS, 2007). The background and relevance of using DRI as the basis for determining outburst TLV has been reviewed and discussed in Black (2018).

In the DRI approach, outburst proneness is regarded by GeoGAS as being directly related to the desorption rate of the coal. Bowen Basin coals (Goonyella Middle and German Creek seams) have higher DRI compared to the Bulli seam and, accordingly, the gas content thresholds are lower. For CH4, the Bulli seam gas content threshold is 9.5 m³/t at a DRI of 900. For the same DRI, the Goonyella Middle seam has a gas content of 7.0 m³/t and the German Creek seam (Middlemount/Tieri) a gas content of 7.7 m³/t (GeoGas, 2007).

The DRI900 method was proposed, based on a review of the Bulli seam threshold for ‘normal mining’, which is effectively equivalent to a TLV for structured coal, and no work was done to establish a method to determine TLV for non-structured coal.

Kidybinski (1980) in Lama (1995) recognised that factors other than gas pressure and gas content play an important role in promoting instantaneous outbursts of coal and gas in coal seams. Outbursts often occur in coal weakened by local geological distortion. Strength variations within the coal, due to tectonic and sedimentary conditions are often greater than strength variations due to variations in gas pressure and desorption phenomena. Kidybinski further suggested that coal strength, coal weakness, may have a greater effect on local outburst hazard than gas pressure and desorption characteristics. These additional, and potentially more significant outburst risk factors are not considered in the GeoGAS DRI900 approach to determination of outburst TLVs in Australian coal seams.
MINING EXPERIENCE ABOVE NORMAL OUTBURST GAS CONTENT THRESHOLD LIMITS

In the years following the 1994 introduction of the Bulli seam TLVs there has been significant advances in directional drilling technology and the standard of management plans used at most mines to identify, assess and control outburst risk. Also, many mines have progressively moved into areas of increased gas content and reduced permeability, where it is becoming increasingly difficult to drain gas below the ‘normal mining’ TLV and mine operators are questioning the appropriateness of the outburst TLVs (Black and Aziz, 2008 and Black et al., 2009).

Several Australian underground coal mines have completed formal reviews of their outburst management plans which led to increasing TLV supported by additional management controls, such as increased drilling density and increased gas content testing. Increased TLV, presented in Figure 10, were approved for Tahmoor colliery in 2003 and West Cliff colliery in 2005. In the years following the changes to the TLV, both collieries operated without an outburst incident. Metropolitan colliery has also reviewed and introduced additional TLVs to allow controlled mining in areas where gas content remains above the 1994 ‘normal mining’ TLV.

Figure 10: Revised outburst TLVs at Tahmoor and West Cliff colliery (Black, 2011)

At Tahmoor colliery, in addition to the Level 1 TLV, below which no restrictions are placed on mining, introduced two additional TLV levels. The Level 2 TLV applies to structured coal and where the measured gas content is greater than Level 1 and less than Level 2, in addition to more intensive drilling and coring, the rate of development advance is restricted to 12 m/day. The Level 3 TLV applies to coal free of geological structures. Where the measured gas content is greater than Level 1 and less than Level 3, in addition to increased drilling and gas content testing, the rate of development advance is restricted to 25 m/day in each heading and out-through to a maximum of 75 m in any 24 hour period. In areas where gas content remains above the defined TLV, normal mining is prohibited and grunching is the only approved development mining method. At West Cliff colliery, in addition to the Level 1 TLV, one additional TLV was introduced. While no restrictions were placed on the rate of development advance, where the measured gas content was between the Level 1 and Level 2 TLV increased drilling, structure identification and gas content testing was required. Where the gas content remained above the Level 2 TLV, normal mining was prohibited and an alternative mining method, such as remote control or grunching, was required.

Figure 11 shows gas test results from areas of the Bulli seam where gas content was above the ‘normal mining’ outburst threshold limit that were mined by non-standard methods without inducing an outburst. The figure shows gas data from areas mined by (a) fully remote controlled continuous miner operation, (b) grunching using conventional shotfiring, and (c) limited rate mining where limits are placed on the maximum hourly and daily rate of advance of conventional continuous mine development operations. In the 15 year period that Tahmoor colliery has employed limited rate mining through structured and non-structured coal, with gas content up to 12.0 m³/t (CH4) and 10.0 m³/t (CO2), an outburst has not occurred (Borg, 2014). Tahmoor has also mined over 3,000 metres of roadways, by grunching due to inability
of conventional pre-drainage to reduce gas content of the coal seam below the original TLV, without inducing an outburst in ‘tight’ coal with gas content up to 14 m$^3$/t (Wynne, 1999, 2000, 2011).

Blanch (2017) raised concerns in relation to the use of limited rate mining, suggesting there was increased risk of an outburst event occurring if mining were to be undertaken in areas where the gas content remained above the 1994 ‘normal mining’ TLV. Tahmoor’s 15 years’ experience mining through such areas, along with similar experience at West Cliff, Metropolitan and other Australian collieries, does suggest (a) the use of limited rate mining is an effective control to mine in areas of increased gas content, or (b) the 1994 TLV are very conservative and the gas content levels in those areas did not present an increased outburst risk, particularly in areas where no geological structurers are present. Further investigations are planned to assess seam gas pressure in advance of development working faces and the impact of mining rate on the seam gas pressure profile. Figure 12 presents gas test results from locations where outburst have occurred in areas of the known outburst risk in the Bulli seam that were mined using remotely operated continuous miner and grunching methods. Like the historical outburst events data presented in Figure 8, the recently acquired outburst data presented in Figure 12 highlights the absence of outburst events below approximately 9.0 m$^3$/t, and does not support the view that coal rich in CO2 is at greater risk of outburst. Further investigations into the impact of gas composition on outburst risk are planned.

CONCLUSIONS

Outbursts represent a major safety hazard to personnel working near the coal face in areas of increased outburst risk. The current approach to TLV have been effective in preventing injury from outburst however increasing evidence, based on operational experience, suggest the current method is overly conservative in some conditions, to the point of adversely impacting mine productivity without delivering significant incremental increase in safety.

While abnormal geological conditions have been linked to all fatal outburst incidents and at least 98% of the non-fatal outburst incidents recorded in Australia, the presence or absence of geological structure is typically not reflected in outburst TLVs in Australian mines. Gas content is recognised as having the most significant impact on outburst risk and gas drainage to reduce gas content to safe levels plays a significant role in control and reduction of outburst risk in Australian underground coal mines.
Investigations into Australian outburst history and mining experience in areas where gas content was above the 1994 Bulli seam outburst threshold limits has provided no evidence that outburst had occurred at gas content levels below approximately 9.0 m$^3$/t, independent of gas composition and geological conditions. Using limited rate mining methods, mining has been carried out, without outbursts, in areas where gas content TLV has been equivalent to 12.0 m$^3$/t (CH$_4$) and 10.0 m$^3$/t (CO$_2$).

While China, Russia and other European countries assess outburst risk through measurement of gas emission rate from fresh cut coal samples, using outburst indices such as $\Delta P$, $\Delta P_{0.60}$, $\Delta P$ Express and $K_I$ Index, Australia is the only country that uses a measure of gas emission from crushed coal during the Q3 gas content testing as the basis for establishing outburst TLVs.

Coal mining practice in Australia requires rapid mining rates to sustain high productivity retreating longwalls which in turn rely on effective systems to identify areas of increased outburst risk and effective treatments to reduce the outburst risk in advance of mining operations.

Control and management of outburst risk, including measures to predict and reduce outburst risk in advance of planned mining, must be effective and continue to support the high safety and production targets of the Australian underground coal mines.

Further work will continue in association with the University of Wollongong to (a) investigate the impact of gas composition, coal toughness and gas pressure on outburst risk, and (b) develop a multi-factor Outburst Risk Index appropriate for assessing outburst risk in Australian mining conditions.

ACKNOWLEDGEMENTS

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GAS DESORPTION RATE OF COAL SEAMS IN ZONGULDAK COAL BASIN AS AN INDICATOR OF OUTBURST PRONENESS

Olgun Esen¹, Abdullah Fisne², Gündüz Ökten³, Dennis Black⁴

ABSTRACT: Outburst is a significant hazard in underground coal mining and may be expressed as a violent ejection of coal and gas from the mining face. Greatest risk of outburst is during initial intersection of an unmined coal seam and during development mining of the coal seam in close proximity to geological disturbances.

Outburst events have been reported during underground mining operations in over 18 countries, including Turkey, for over 150 years. In the Zonguldak Coal Basin, located on the Black Sea coast of North West Turkey, 90 outbursts were recorded over 44 years, between 1969 and 2013, resulting in 374 fatalities.

To protect the mine workings from the outburst hazard, the outburst indicators, \( \Delta P_{0-60} \), \( \Delta P_{\text{express}} \) and the \( K_T \) index, have been investigated to evaluate potential application to predict outburst prone areas. The study of 166 coal samples collected from the three (3) coal seams, Acilik, Sulu and Cay seams, mined at Kozlu and Karadon collieries in the Zonguldak Coal Basin, found the results of \( \Delta P_{0-60} \), \( \Delta P_{\text{express}} \) and the \( K_T \) index ranged between 2 to 26 mmHg, 0.16 to 0.76 bar, and 0.57 to 0.79 respectively. These results were compared with threshold limit values reported in previous studies to identify areas of potential increased outburst risk.

INTRODUCTION

An outburst is a sudden release of gas and coal under pressure from a working face area (Black, 2011). Various theories have been presented regarding factors that contribute to the occurrence of coal and gas outbursts. Factors that may affect outburst potential include tensile strength of coal, gas emission rate, gas pressure gradient, moisture content and the magnitude of local stresses (Lama, 1995). Specifically, high levels of seam gas near geological structures have been identified as a major contributing factor in the coal and gas outburst phenomenon (Lama, 1995). In Turkey, methane (CH4) is the dominant seam gas and has been associated with past outburst events. Carbon dioxide (CO2) and variable concentrations of CH4 and CO2, in combination with the ejection of fine coal particles, have also been released in outburst events reported in Australia, Canada, China, Czech Republic, France, Poland (Beamish and Crosdale, 1998; Lama and Bodziony, 1998; Lama and Saghafi, 2002; Liu et al., 2008; Aziz et al., 2011).

Despite much research, the coal and gas outburst phenomenon is not well understood and many researchers continue to investigate the occurrence mechanism and prediction techniques. In Turkey, coal and gas outburst events were first recognised and reported in 1969 and in the 44 years to 2013, 90 outburst events were recorded, all events occurring in the Karadon and Kozlu collieries located in the Zonguldak Coal Basin. Fisne and Esen (2014) reported the highest number of outbursts occurred during mining of three seams at the two collieries; 20 outburst events in the Acilik seam, 15 outbursts in the Cay seam, and 13 outbursts in the Sulu seam.

The two methods used for outburst prevention in Turkish coalmines are (a) local drilling of boreholes to reduce gas pressure, and (b) protective seam mining. However, further work is needed to identify accurate and reliable methods to locate areas of increased outburst risk, where outburst prevention efforts can be concentrated. Four factors are considered to have the greatest impact on outburst propensity, the most significant being gas content/gas

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pressure, have been presented as an outburst risk matrix by Black (2017) and Black et al., 2009 (Figure 1).

![Outburst Risk Matrix](image)

**Figure 1: Outburst risk matrix (Black, 2017).**

Bodziony and Lama (1996) described outburst prediction indices, such as $\Delta P_{0-60}$, $\Delta P_{\text{express}}$, and the $K_T$ index, used with varying levels of effectiveness by different countries, to identify areas of increased outburst risk. While these prediction indices, related to initial gas desorption, must be used in conjunction with other parameters such as coal properties, coal seam gas content and stress conditions, to fully assess outburst risk, it is important to recognise and accept the importance of gas desorption rate from coal while investigating the outburst prone coal seams.

In this study, $\Delta P_{0-60}$, $\Delta P_{\text{express}}$ index, and $K_T$ index are determined for coal samples collected from the Acilik, Sulu and Cay coal seams in Kozlu and Karadon collieries and the results are assessed to identify potential outburst prone conditions in each seam.

**STUDY AREA**

Zonguldak Coal Basin, located on the Black Sea Coast (Figure 2), is the only bituminous coal basin in Turkey. Zonguldak Coal Basin is the main part of the Upper Carboniferous bituminous coal basin, much of the bituminous coal mining has thus been concentrated in the Zonguldak Basin (Karayigit, 2001). The coal seams are located in a Carboniferous deltaic sequence of Westphalian-A age. The coalfield has a complex and hard geological condition, first by Hercynian and later by Alpine orogenesis resulting in folding and faulting of strata (Okten, et al., 1995). The Carboniferous coal-bearing sequence of the Zonguldak basin contains the Namurien Alacaazgi Formation, Westphalian-A Kozlu Formation and Westphalian B-D Karadon Formation (Gurdal and Yalcin, 2000).
Mining activities in the basin started in 1848 and have continued in the region for over 160 years. Several national and international companies operate coal mines in the basin. The mining area is 6885 km$^2$ and mining depth continues to increase. Coal is produced from five collieries, shown in Figure 3, Armutçuk, Kozlu, Uzulmez, Karadon and Amasra that are operated by the Hard Coal Agency of Turkey (TTK). In Kozlu colliery, mine workings have extended out below the Black Sea and have reached depths of 1200 metres. The saleable production is relatively low by Australian standards, totalling 948,573 tonnes in 2015, 908,107 tonnes in 2016, 2017 production year-to-date is approaching 830,000 tonnes. TTK employs 9,000 people and 6,000 of those employees work in the underground mines (TTK Annual Report, 2017). Increasing gas content and high gas concentration in working places has been identified as a contributing factor to reduced coal production from the TTK collieries. While the Acilik, Sulu and Cay seams are recognised as having the highest propensity for coal and gas outbursts, they contain the highest quality coal and are the target for intensive coal production in the basin.
content, and stress, to thoroughly assess outburst risk. The following three indices have been investigated:

- $K_T$ index
- $\Delta P_{0-60}$ index; and
- $\Delta P_{express}$ index.

**$K_T$ index**

This index is a measure of the change in desorption rate of a coal sample. Plotting measurements of gas desorption (cm$^3$/min.kg) relative to desorption time (min) on a ln-ln scale, the slope of the curve represents the $K_T$ value. The method of sampling consists of drilling holes and collecting fractions of particles in the range 0.40 to 0.63 mm and the mass of the sample depends upon the capacity of the equipment. Lama and Bodziony (1996) suggest the critical value $K_T < 0.645 \pm 0.035$ is considered normal and for outburst conditions, $K_T$ should be at least 0.75. Lama and Bodziony also suggest the critical $K_T$ value relates to a gas content of 9.0 m$^3$/t.

Ökten (1983) proposed the $K_T$ index categories, listed in Table 1, to classify outburst prone coal seams.

**Table 1: $K_T$ index classification for coal and gas outbursts.**

<table>
<thead>
<tr>
<th>Category</th>
<th>$K_T$ Range</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.75 to 0.82</td>
<td>Potential of Outburst</td>
</tr>
<tr>
<td>2</td>
<td>0.82 to 0.88</td>
<td>Risk of Outburst</td>
</tr>
<tr>
<td>3</td>
<td>&gt; 0.88</td>
<td>High Risk of Outburst</td>
</tr>
</tbody>
</table>

**$\Delta P_{0-60}$ index**

Lama and Bodziony (1996) referred to work of Ettinger et al. from 1953, which presented an alternative to measuring gas desorption as a percentage of gas sorbed at 1 atm pressure, which involved measuring gas pressure build up in an enclosed chamber of definite dimensions, expressed as "gas emission index".

The $\Delta P_{0-60}$ index, which is a measure of pressure rise in the initial 60 second period has been used to predict outburst prone coal seams in a number of countries (Lama and Bodziony, 1996). The method involves drilling 3.0 metre long holes in advance of development and longwall faces, with boreholes spaced every 15 metres across a longwall face, as shown in Figure 4. Cuttings are collected from the last half metre of each hole, from 2.5 to 3.0 metres, and sieved. In testing bituminous coal, a 3.0-gram sample of 0.25 to 0.50 mm particle size is enclosed in a sealed chamber within 90 seconds of drilling and the change in gas pressure (mmHg) is recorded over a period of 60 seconds.

![Figure 4: Placement of holes for sampling for $\Delta P_{0-60}$ determination (Lama and Bodziony, 1996).](image-url)
The critical value of $\Delta P_{0-60}$, above which is considered a sign of imminent outburst risk, is 15 mmHg (~20 mbar) (Lama and Bodziony, 1996). Vandeloise (1964) proposed $\Delta P_{0-60} = 20$ as the lower limit of outburst danger in Belgian coal whereas in the Cevennes coalfield, the $\Delta P_{0-60} = 14$ was reported to be the lower limit of outburst danger. The classification for $\Delta P_{0-60}$ index values defining outburst risk, presented by Lama and Bodziony (1996), has been listed in Table 2.

Table 2: $\Delta P_{0-60}$ index classification for coal and gas outburst

<table>
<thead>
<tr>
<th>Category</th>
<th>$\Delta P_{0-60}$ Range (mmHg)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 to 15</td>
<td>Not Prone to Outburst</td>
</tr>
<tr>
<td>2</td>
<td>15 to 30</td>
<td>Slightly Suspect</td>
</tr>
<tr>
<td>3</td>
<td>30 to 45</td>
<td>Suspect to Outburst</td>
</tr>
<tr>
<td>4</td>
<td>45 to 60</td>
<td>Dangerous</td>
</tr>
<tr>
<td>5</td>
<td>&gt; 60</td>
<td>Highly Dangerous</td>
</tr>
</tbody>
</table>

Lama and Bodziony (1996) report that the $\Delta P_{0-60}$ index value measured from testing dull coal is typically higher than from bright coal which they suggest may be due to dull coal having lower diffusivity. The $\Delta P_{0-60}$ index value is affected by ash content of the coal sample and Lama and Bodziony presented the modified value to account for ash that is given by the equation:

$$\Delta P_{0-60} (afb) = \frac{\Delta P_{0-60}}{1 - Ash\%}$$

$\Delta P_{\text{express}}$ index

Paul (1977) modified the $\Delta P_{0-60}$ index testing method to provide a new index value which was named $\Delta P_{\text{express}}$. The method involved in determining the $\Delta P_{\text{express}}$ index requires a coal sample of about 70 g in the range of 0.25 – 0.5 mm for bituminous coal to be sealed in a chamber. The sample is evacuated for 2 minutes and then methane is allowed into the sealed chamber to raise the pressure to 200 kPa. The chamber is then immediately connected to a manometer and the change in pressure over time is recorded. The gas pressure recorded on the manometer after a period of one (1) minute gives the $\Delta P_{\text{express}}$ index (Lama and Bodziony, 1996).

Paul (1977) presented data from testing anthracite coal that indicated a correlation between the $\Delta P_{0-60}$ and $\Delta P_{\text{express}}$ index values. In that example, the critical value of $\Delta P_{0-60}$ index, 15 mmHg, was found to correspond to a $\Delta P_{\text{express}}$ index value equal to 0.45 bar (45 kPa) for anthracite. This threshold limit can be used to compare the coal seams in terms of outburst proneness.

RESULTS AND DISCUSSION

In this study, coal samples were collected from the Acilik, Sulu and Cay seams. As the majority of outburst events have been recorded in these three coal seams mined in the Zonguldak Basin, it is suggested they represent the greatest outburst risk. The gas desorption rate from each seam has been investigated by measurement of the $\Delta P_{0-60}$, $\Delta P_{\text{express}}$, and $K_T$ indices. According to Okten (1983) and Esen (2013), data from 166 coal samples were combined and compared with their threshold limits to determine outburst propensity. Coal samples were taken only from Kozlu and Karadon Collieries in different mining areas such as from longwall face, raise and gateway. The results of desorption rate measurement are listed in Table 3. Measurement of the $\Delta P_{0-60}$ and $\Delta P_{\text{express}}$ index was completed on a total of 118 coal samples and the $K_T$ index was measured on 48 coal samples. The results obtained from the gas desorption index measurement were found to range between 0.16 to 0.76 bar for $\Delta P_{\text{express}}$, 2 to 26 mmHg for $\Delta P_{0-60}$, and 0.57 to 0.79 for $K_T$ index.
Table 3: Gas desorption rate data.

<table>
<thead>
<tr>
<th>Indicator</th>
<th>Number of Samples</th>
<th>Index Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Acilik</td>
<td>Sulu</td>
</tr>
<tr>
<td>$K_T$ index</td>
<td>22</td>
<td>14</td>
</tr>
<tr>
<td>$\Delta P_{60}$ (mmHg)</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>$\Delta P_{\text{express}}$ (bar)</td>
<td>22</td>
<td>22</td>
</tr>
</tbody>
</table>

The $K_{T}$ index is considered as important in assessing outburst propensity of coal seams in underground coal mining. Testing to determine the $K_{T}$ index was completed on 48 coal samples; 22 samples from the Acilik seam, 14 coal samples from the Sulu seam, and 12 samples from the Cay seam. The results of the $K_{T}$ index value of each seam and their relationship with $K_{T}$ outburst threshold limit were summarised in Figure 5.

From testing, two (2) coal samples were found to have $K_{T}$ index values of 0.75 and 0.79, which based on the $K_{T}$ index outburst classification, presented in Table 1, suggests the area where those samples were collected from Acilik seam may be potentially outburst prone. In addition, three (3) coal samples are being very much closer to the critical $K_{T}$ threshold limit with a value of 0.730, 0.730 and 0.740, which were taken from Acilik seam. Cay and Sulu Seam have lower $K_{T}$ index values and have no outburst potential.

![Figure 5: Results of $K_{T}$ values of each seam and their relationship with $K_{T}$ outburst threshold limit.](image)

The coal being mined in the Zonguldak Coal Basin is bituminous and there is a need to establish a process that can be applied to accurately and reliably describe a threshold limit value to classify outburst prone areas of the coal seams. The data from Esen (2013) and Okten (1983) was combined on the plotted graph (Figure 6). An outburst Threshold Limit Value (TLV) corresponding to the critical value of 15 mmHg for the $\Delta P_{60}$ index, as proposed by Lama and Bodziony, was applied to the index data representing the testing on coal samples from the Acilik, Sulu and Cay seams and the corresponding critical value of $\Delta P_{\text{express}}$ index was found to be equal to 0.32 bar. It is therefore suggested that the $\Delta P_{\text{express}}$ index be used as an indicator of outburst risk for mining in Acilik, Sulu and Cay coal seams with a TLV equal to 0.32 bar (35 kPa) established as the critical value.
Figure 6: The relationship between $\Delta P_{0-60}$ index and $\Delta P_{\text{express}}$ index.

Figure 7, Figure 8 and Figure 9 present the relationship between the $\Delta P_{0-60}$ index and the $\Delta P_{\text{express}}$ index values determined for the Acilik seam, Sulu seam and Cay seam respectively. From the 22 coal samples tested from the Acilik seam, presented in Figure 5, eight (8) coal samples lie above the proposed TLV. In the outburst history of Karadon and Kozlu collieries, which have experienced most outbursts in the region, the greatest number of outburst events has occurred in the Acilik seam.

The gas emission data indicates higher gas desorption from the Sulu seam relative to the Cay seam. The data collected from testing the 22 samples from the Sulu seam, presented in Figure 7, shows that only two (2) samples were approaching the proposed TLV. The results suggest that in the current mining areas, particularly those areas sampled in the Cay seam, there is presently a low risk of outburst.

In conclusion, from experience and data collected and presented, the Acilik seam has been confirmed as the most outburst prone coal seam being mined in the Zonguldak Coal Basin. Testing of coal samples from the Sulu seam indicated conditions are approaching potential outburst risk levels. The results of testing coal sample from the Cay seam indicates that present mining areas are at comparatively low risk of outburst.
Figure 8: The relationship between $\Delta P_{0-60}$ index and $\Delta P_{\text{express}}$ index for Sulu Seam

![Figure 8: The relationship between $\Delta P_{0-60}$ index and $\Delta P_{\text{express}}$ index for Sulu Seam](image)

Figure 9: The relationship between $\Delta P_{0-60}$ index and $\Delta P_{\text{express}}$ index for Cay Seam

![Figure 9: The relationship between $\Delta P_{0-60}$ index and $\Delta P_{\text{express}}$ index for Cay Seam](image)

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Grateful thanks to Professor Naj Aziz for his helpful and fruitful support and giving us the chance for presenting this paper. Furthermore, the paper benefited from valuable support given by the Scientific and Technological Research Council of Turkey. The authors also would like to express their gratitude to Polyak Eynez Mining Corporation in Soma, Manisa, Turkey for providing coal samples and performing the surface exploration drillings in the Soma Eynez coal basin.

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DETERMINATION OF GAS EMISSION IN THE MINING LIFE CYCLE

Patrick Booth¹, Heidi Brown, Jan Nemcik, Ting Ren

ABSTRACT: Design of optimal gas drainage and gas management programs throughout the mining cycle is contingent upon a thorough understanding of the gas emission mechanisms specific to the geological and operational conditions in a particular location. As mining progresses from exploration, through development and into production, the resolution of data typically used for gas emission prediction improves spatially and with respect to time.

Quantification and management of risk associated with sudden gas release during mining (outbursts) and accumulation of noxious or combustible gases within the mining environment is reliant on gas emission prediction, which is spatially relevant and applicable to the mining stage being undertaken. Using iterative spatial interpolation techniques, appropriate resolution gas emission model input data may be used to continually improve both the resolution and accuracy of model outputs and also determine triggers where model recalculation is required.

Proposed techniques are validated through a case study of gas core samples obtained from two southern Sydney basin mines producing metallurgical coal from the Bulli seam over a period of 10 years. Alignment of data in various geospatial and extraction time-based context, including relationships to hydrological features and geological structures, combined with experimental results assessing the influence of changes in confining stress and gas pressure, appear to align with modelled outputs and recent historical gas emission data. The results suggest variability and limitations associated with the present traditional approaches to gas emission prediction and design of gas management practices may be addressed using predictions derived from improved spatial datasets, and analysis techniques incorporating fundamental physical and energy related principles.

INTRODUCTION

Underground mining methods account for approximately 20 per cent of total black coal production in Australia (Geoscience Australia 2015). In NSW, metallurgical coal is exclusively mined from the Illawarra coal measures in the southern region of the Sydney basin. Collocated with these coal reserves are significant quantities of methane (CH₄) and carbon dioxide (CO₂) gas (Geoscience Australia 2015). Gas reserves are not limited to economically recoverable coal seams, but also include coal measures and other porous stratigraphy both above and below the working seam (Karacan et al. 2011). Emission predictions are essential for the quantification and management of risk associated with sudden gas release during mining (outbursts), and accumulations of noxious or combustible gases within the mining environment. Unexplained variation in gas character rightly requires conservative mining practices to manage such risks (Balusu et al. 2010). In many cases, risks are identified later in the mining cycle where remedial action is typically more expensive and is more likely to incur production delay or loss. Gas core samples from two southern Sydney basin mines producing from the Bulli seam have been analysed in various geospatial contexts including relationships to hydrological features and geological structures. Improved spatial datasets, particularly those containing a vertical dimension and derivatives thereof, may be applied to prediction and management of gas emission using fundamental principles. The application of the physical and spatial techniques described enhance the potential future use of high volume and high resolution real time measurement data for proactive management of gas emission risk much earlier in both the gas and mining life cycle. The improved resolution and definition in the prediction of site specific transient gas emission character, in terms of source location, quantity, composition, flow path and timing is acknowledged by several authors as critical for maintaining current production rates. (Karacan et al. 2011; Packham et al. 2011; Wang et al.

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2011). Gas emissions will increase well beyond the practical management capacity of ventilation and current pre and post drainage systems at several Australian underground coal mines (Balus et al. 2010). Hence the traditional approach of increasing ventilation quantity is unlikely to be sustainable due to practical constraints such as roadway area and maximum air velocity therein. The identification and use of gas management controls which are fundamentally based and incorporate improved spatial and time resolution will not only make mining safer, delivery of this outcome will reduce interruptions for reasons of safety management and lift overall coal and energy productivity.

**HISTORICAL GAS EMISSION PREDICTION**

The prediction of methane emissions arising from underground coal mining has been the subject of extensive research for several decades (Creedy 1993; Curl 1978; Karacan 2008; Lunarzewski 1998) and techniques range from simple geometric models to modern finite element models (Ashelford 2003; Guo et al. 2012). Despite improvement in computation processing power and speed over this time period, calculation techniques remain empirically based and are hence limited to the origin of information in both application and resolution. Gas emissions due to mining extraction are transient and a complex function of the in-situ resource character, the space where in-situ character and gas equilibrium is affected by extraction, the degree to which character and equilibrium is affected, and the system response (Lunarzewski 1998). In order to simplify the calculation process of most current prediction techniques, key inputs for gas content, material properties and spatial attributes are generally either provided as input variables at low resolution, held constant, or neglected altogether. Of the many prediction techniques available, the Flügge technique continues to be used for the purpose of total specific gas emission calculation at many Australian mines (Black 2011; Meyer 2006). However, limitations in describing spatial and time based gas emission character with any resolution renders this technique ineffective for design of gas drainage programs. Evidence provided through finite element analysis and micro seismic observations suggest the triangular prism representation is only valid in specific geological conditions and does not cater well for changes in either geology or operational practices (Kelly et al. 1998). Limitations of the technique also extend to neglecting from calculation of fundamental mechanisms driving gas emission behaviour from coal and surrounding strata (Barker-read andand Radchenko 1989; Gray 1987; Lama andand Bodzioni 1998). The significance of localised cleat and joint geometry and net effective stress in the control of fluid movement is similarly neglected. All gas emission prediction techniques require initial measurement of gas content, and the resolution of this measurement may range from kilometres to sub-metre depending on the mining cycle stage. A detailed description of the process for measurement of gas content and its contributing components may be found in AS 3980 (Standards 2013). Limitations of some of the measurement techniques used, specifically including assumptions of the timing of initial desorption and the lost gas component $Q_1$, are discussed further by Saghafi (2016). The GeoGAS Longwall “Pore Pressure” model described by Ashelford (2003) took account of many gas reservoir and geological parameters of coal seams and allowed variation of mining operations in arriving at a gas emission value. The model relies upon measured gas reservoir properties for the determination of gas release such as; measured gas content ($Q_m$), gas desorption rate, gas composition, gas sorption capacity, seam thickness and mineral matter above and below the working section, pore pressure and coal and sandstone porosity. The model parameters and how they are measured are described by Williams et al (2001). The advantage of this model over prior techniques was its’ ability to accurately predict the magnitude of gas emission from the floor seams below the Bulli seam in the southern Sydney basin. This was due to the significant deformation and order of magnitude changes in horizontal and vertical stress in the floor strata recognised and displayed by finite element software. Whilst the pore pressure model remains the most adaptive and fundamentally based calculation of gas emission for longwall operations, the input assumptions limit the application of this technique to the increasing spatial and time resolution required for design of gas drainage programs. The availability of increasing computational processing capability has enabled the management of the increased size and complexity of the data available for gas emission analysis in recent years. Studies including those by Karacan (Karacan 2008; Karacan andand Goodman 2012; Karacan andand Olea 2014) used statistical, Principle Component Analysis (PCA) and Artificial Neural Network (ANN) based approaches to predict the ventilation methane emission
rates of U.S. longwall mines. Critically, all techniques, which involve the use of large historical data sets for gas emission prediction by analysis using statistical, PCA or ANN approaches rely on a fundamental assumption that input conditions will not materially change. Model outputs are based on fundamental scientific principles, however the model design and structure limit the ability for its use in locations where input conditions change rapidly, such as adjacent to or across geological structures. Comparison of the output of various prediction models is difficult due to lack of a common gas, material and spatial datum reference and also for the reasons discussed in Jensen et al (1992).

RELEVANT GAS FUNDAMENTALS USED

Gas Generation

Coalbed or coal seam gas are general terms used to describe gases contained within coal measures that are generated as part of coalification and other geological and hydrogeological processes (Flores 1998). Similar to the creation of coal itself, coal bed gas generation pathways are also dependent on fundamental physical and chemical characters and changes in both the level and form of energy within the environment. Coal bed methane can be classified as either biogenic or thermogenic in origin (Moore 2012). Biogenic methane is generated at low temperature by anaerobic microbes (methanogens) when coal beds are exposed to groundwater recharge after basin deformation. Two significant factors must be carefully considered in the characterisation of the origin of biogenic gas. Firstly, for carbon dioxide reduction to methane, Hydrogen must be present. Secondly, in addition to the methane, the two-part acetate fermentation process also produces CO₂ (Burra et al. 2014). The flow pathway of water is therefore an important factor in characterising gas reservoir conditions. The relative rate of change of coal seam gradient and orientation hence provide information on available potential energy under the influence of gravity. The effect of gravity on hydrogeological and material deposition character has remained constant over geological time. Thermogenic gas is generated at high temperature during late stage coalification and generally contains heavier carbon isotopes than biogenic gas. The results described from (Moore 2012) indicate that the first gas generated via thermogenic processes is CO₂ at approximately 50 °C. Above this temperature, increasing amounts of hydrocarbons (methane, ethane and higher) and nitrogen are produced at maximum volume at approximately 150°C. At higher temperature, gas generation reduces, producing a parabolic maximum gas volume trend with temperature and/or rank. Such parabolic gas content trends have been reported from a number of Australian Basins including the southern Sydney basin which is the subject location of this research (Faiz et al. 2007).

Gas Storage

Over ninety per cent of gas storage in coal occurs by physical adsorption to the surface of the coal matrix, including the surfaces of all internal pores and cleats or fractures (Flores 1998). The remaining is free gas, which may also reside within internal pores depending on pore geometry, and also within cleats or fractures. Adsorption concepts between gas and a solid surface are usually described in terms of isotherms, where the amount of adsorbate on adsorbent is shown as a function of pressure at constant temperature as depicted for three gases (CO₂, CH₄ and N₂) at one of the study sites in Figure 1. The figure also demonstrates the range of potential variation in sorption capacity from upper, middle and lower sections within the 2-3 metre thickness of the Bulli seam.
Gas composition is a fundamental controlling variable in determining total possible sorption capacity due to the relative size, structure and energy levels of relevant gas molecules, particularly CH$_4$ relative to CO$_2$ (Booth et al. 2016; Mosher et al. 2013). However, the availability of adsorption sites or coal internal surface area remains the key limiting parameter. The coal structure hence sets the adsorption and desorption character, which also changes with gas type, but this does not necessarily mean that a coal of certain properties and whose sorption capacity is described by a particular isotherm actually contains that amount of gas for a given volume (Black 2011). The ratio between actual measured gas content and the theoretical sorption capacity is known as the degree of saturation and is expressed as a percentage. Lower insitu degree of saturation is an indicator that other mechanisms, such as lowering of hydrostatic pressure through fluid movement, have potentially allowed gas to migrate or otherwise be released from the coal after initial gas generation. Gas storage capacity is hence defined by the combination of gas composition and coal properties and structure. Using fundamental physical, chemical, energy and geometric relationships, it is postulated that for gas emission prediction, dynamic response of the gas reservoir to mining extraction can be reliably predicted using higher resolution input spatial parameters, and measured coal property data which is largely available through proximate characterisation parameters such as rank, carbon content, macerals, and moisture content. The range of possible variation in coal properties within the Bulli seam at a single location is demonstrated in Table 1. Coal structural properties are also unlikely to remain constant over a given mining horizon, but rather be significantly influenced by the landscape at time of deposition. Hence these properties are influenced by spatial factors, which may be either measured directly or reliably interpolated using fundamental spatial relationships. Mineralisation also influences internal structure, geometry, pore availability to gas adsorption, ability of gas to flow, shrinking and swelling, gas content, gas recoverability, and potential for enhanced gas recovery. (Flores 2013) Experimental evidence closely correlates increasing coal rank with higher proportions of micropores (Mosher et al. 2013). An increase in micropore distribution per unit of coal volume also increases surface area available for gas adsorption, hence explaining observed experimental increase in gas storage capacity with coal rank, and increase in rate of change of volumetric capacity per unit pressure change as described by Kim in the review by Moore (Moore 2012).

Gas Flow

The movement of gas molecules through either other gases, fluids or solids are described by Fick’s Laws (Saghafi 2016). On reducing spatial component of both equations (dx), it becomes more probable that molecules will be subjected to much larger external energy forces (e.g. pressure gradients) in shorter timeframes.
Darcy's law is an expression of conservation of momentum and describes a proportional relationship between the instantaneous discharge rate through a porous medium, viscosity of the fluid and pressure drop over a given distance. This equation can also be solved for permeability, allowing for relative permeability to be calculated. In practice, this measurement is difficult and expensive to complete in situ, but is the only method of obtaining a true permeability result which reflects the reservoir conditions (Gray 1987).

In the case of coal, permeability is a complex, multi-dimensional function of several influences such as width, length, height, aperture spacing, frequency or density, and connectivity of cleats or fractures (Flores 1998). Many of these influence functions are non-linear, however, they have components that can be either readily measured directly or indirectly or otherwise grouped without affecting materially affecting calculation results. Changes in permeability in coal may be summarised into two main components; the effective stress effects, and the shrink and swell strain effects on the coal matrix with desorption or adsorption which may increase or decrease relative permeability (Cai et al. 2014). Coal composition hence controls a broad range of gas reservoir properties including gas adsorption capacity, gas content, porosity, permeability and gas transport.

Table 1: Coal properties at a single location within the Bulli seam

<table>
<thead>
<tr>
<th>Sample</th>
<th>Proximate analysis (wt.%)</th>
<th>Coal grain density (kg/m$^3$)</th>
<th>Intraparticle porosity (%)</th>
<th>Surface area (m$^2$/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>1.35</td>
<td>12.53</td>
<td>17.98</td>
<td>68.14</td>
</tr>
<tr>
<td>Middle</td>
<td>1</td>
<td>10.53</td>
<td>23.55</td>
<td>64.92</td>
</tr>
<tr>
<td>Bottom</td>
<td>1.15</td>
<td>25.88</td>
<td>15.06</td>
<td>57.91</td>
</tr>
</tbody>
</table>

All Proximate analysis on percentage by weight air dried basis;
Surface area data is calculated based on the pores whose diameter above 30nm.

CALCULATION OF RELEVANT SPATIAL PROPERTIES

The fundamental nature of the physical and chemical interactions between the principal components of coal, coal seam gases and other substances found in the mining environment are significantly influenced by the various forms of energy applied over time. However, the potential energy involved in sedimentary deposition, gas generation and flow is of particular relevance to analysis of gas emission at higher resolution. An overview of the process for calculation of the relevant spatial properties is depicted in Figure 2. In the absence of vertical dimension data specific to the location of gas core samples, alternate sources of vertical information or interpolation can be used to inform predictive modelling of gas emissions.

Figure 2: Process for calculation of spatial properties.

Derivation of the elevation surface

The goal of the first stage of model development is to obtain the best possible representation of the Relative Level (RL) of the floor of the coal seam in the subject area using a common datum. To achieve this, original sources of data included but were not limited to, manual survey, drilling records and seismic interpretation. Ideally, all input data is provided in the form of three-dimensional points, however this is not always available. Care must be taken to ensure the use of a common reference datum for all available location and level measurements entered as input data. The Map Grid of Australia (MGA) Zone 56 and
Australian Height Datum (AHD) were selected as the common reference datum for all location and level information used, and several data sources initially required conversion to this datum. The dimensional convention used throughout this study is X related to longitudinal co-ordinates, Y related to latitudinal co-ordinates, and Z related to height or RL co-ordinates. Although sources of RL data may include contours, these are generally previous interpolations of X, Y, and Z point data sources. To allow for future model prediction, development and improvement in real time location measurement technology, all RL input data sources were converted to X, Y and Z point data before proceeding to the next stage.

Selection of the interpolation technique suitable for creation of the elevation surface used in this study considered a range of selection criteria. Input data and processing constraints, and future use of the interpolation outputs in later model processes received higher weighting in the assessment process. Of the many techniques described and compared in the literature (Li and Heap 2008), the spline with barriers interpolation was selected due to the use of exact measured point data as input data, the ability of the technique to manage known abrupt changes in level (e.g. geological faults), the maintainability of the interpolation process with updated input data, and the ability to balance competing requirements for processing time and output resolution.

The result of the interpolation is a raster surface of configurable grid cell size, using barriers, from measured points using a second-order minimum curvature spline technique which is more reflective of natural depositional processes. The barriers are entered as polyline features, and the resulting smooth surface is constrained by the input barrier features. Input datasets may also have several points with the same x and y coordinates. An important feature of this technique is that if the values of the points at the common location are the same, they are considered duplicates and have no effect on the output.

Several tests were undertaken of total surface calculation area versus raster resolution (individual cell size) and processing time. The final selected Digital Elevation Model (DEM) configuration for the two study sites covered an area of approximately 200 km² each, contained multiple barrier features of both two and three-dimensional data types, at a 1 m x 1 m resolution and processed in approximately 10 minutes. This configuration was deemed acceptable for future use and maintainability of the modelling process, and considering development to multi-stratum environments. Figure 3 provides an example three-dimensional DEM representation of the Illawarra coal measures viewed from the north looking south.

The Bulli seam is the uppermost seam in the sequence displayed in light grey, conformably overlying up to 5 other seams with ten times vertical exaggeration. Surface elevations appear in green, generally above the AHD zero reference shown in blue. The study area is shown in light tan with relevant geological structures shown in red.

Figure 3: Example three-dimensional DEM of study area.
Spatial parameters derived from elevation surface

Spatial parameters deemed essential for calculation of gas emission character across the mining environment, and potentially calculated at the required higher resolution (i.e. each 1 m x 1 m cell) are: the vertical dimension Z in metres (AHD), the maximum slope in degrees (0°- 90°) the aspect in degrees (0°-360°), and the curvature in metres per metre, overall and then separately in plan and in profile. The process used for DEM calculation allows selection of the appropriate cell size commensurate to the resolution and sensitivity of the input data to the calculation outcome, and the limited benefit of recalculation if no material variation in input data is observed. Calculation of the above italicised terms all involve the evaluation of the cell in consideration against each of up to eight of its neighbouring cells in the X.Y plane. The slope is a representation of the maximum rate of change of elevation (Z) with respect to both X and Y. The aspect represents the orientation of the slope, where values near 0° and 360° indicate a north facing area, 90° an easterly facing, 180° a south facing, and 270° a west facing area.

The calculation of curvature is the derivative of the slope (i.e. d²z/dxy²) using a similar process to the slope calculation, but using the slope value for each cell as the input to the curvature calculation. By definition, the curvature at a point with zero slope (flat) will also be zero. This phenomenon may be used to determine areas likely to retain fluid, also known as sinks. Curvature may be further defined into profile and plan curvatures, which are useful for describing the acceleration or deceleration of flow paths in the case of the profile curvature, or convergence or divergence of flows in the case of plan.

Application of spatial parameters

Over 2500 gas core sample locations from two mine sites were initially provided in the form of AutoCAD drawing files, complete with two-dimensional X and Y co-ordinates. Two-dimensional drilling trajectories to obtain the core samples were also provided in most cases. Laboratory sample analysis results containing a range of gas properties were provided in the form of MS Excel spreadsheets. Gas parameters included gas content, gas composition and concentration, and desorption characteristics with a unique reference to a sample or core identification number.

The first stage of assessment of spatial parameters involved referencing AutoCAD two-dimensional location information for each unique sample to the laboratory analysis results. Once gas core sample locations were confirmed as two-dimensional X, Y points, the next stage of assessment involved the allocation of all previously calculated values for elevation, slope, aspect, and curvature to each of the gas samples. The output of these previously calculated spatial parameters was, in each case, a raster surface of 1m x 1m resolution. Subject to the quality and resolution of input data, similar interpolation techniques may be applied for the calculation of the distance and direction to geological structure. Structures may include faults, dykes or other anomalies and may be represented as either a two or three-dimensional features. It is anticipated that inclusion of full three-dimensional structure geometry, complete with appropriate attributes for the description of other geological observations, may allow a pathway for stress magnitude and orientation data to be included in overall gas emission modelling without exhaustive computational overhead. Finite element stress modelling for the southern Sydney basin described by many researchers appears to be capable of providing such data (Heritage et al. 2017; McGregor 2003; Tarrant 2006).

OBSERVATIONS AND RESULTS

As existing mining threshold limits are determined primarily by measured gas content and gas composition, these dependent variables were considered initially. Assessment of the full dataset's gas content result by X, Y and Z location did not reveal any significant first order linear trend. However, assessment of gas composition using CO₂ concentration by X, Y and Z dimensions revealed a localised trend with respect to the Z dimension at each individual mine as shown in Figure 4.
Localised trends appeared at each mine with increasing CO₂ concentrations being observed downslope of higher CH₄ concentrations and geological features. Increasing observed gas content with CO₂ concentration aligns with the experimentally determined isotherms for the mine (Figure 1), recognising that coal structural properties and hence sorption capacity is also likely to vary relative to many spatial parameters. Difference in localised seam hydrostatic pressure may also account for such observations, however in the absence of in situ pressure measurement, this could not be confirmed.

The introduction of further independent spatial variables for slope, aspect, curvature and geological structures suggested a strong dependence between higher gas content and areas where localised fluid accumulation or flow restriction was likely to occur. The number of core samples taken in these areas over an extended time period, combined with the number of gas drainage holes drilled in the immediate area, suggest that these areas were also difficult to drain.

An example of these areas within Mine A is depicted in Figure 5. The gas composition of this particular area was greater than ninety percent CO₂, however the dependence between areas of likely fluid accumulation and higher gas content appeared to be independent of gas composition. Other areas of Mine A with higher CH₄ composition also demonstrated a similar relationship. The seam reservoir gas pressure for this area was estimated to be in the order of 3 MPa.

Datasets collected from each mine include features and attributes, which allow calculation of gas drainage quantities and timing. Spatial relationships between various attributes may be assessed using the same process as described previously. Furthermore, the inclusion of inclination data from Drill Guidance Systems (DGS) allows direct calculation of effective horizontal and vertical permeability and hence dynamic drainage rate.
Figure 5: Example gas content distribution analysis at Mine A using aspect and slope parameters.

Data from Mine B also suggests a similar strong relationship between spatial characteristics and higher gas content. At this mine, such spatial relationships also appeared to be independent of gas composition. Areas dominated by higher CO$_2$ concentration were laterally separated from areas of higher CH$_4$ concentration by over 2000m. However, a similar localised trend of higher CO$_2$ concentrations downslope of geological features and higher CH$_4$ concentrations was observed.

In summary, each observed high gas content sample location exhibited one or more of the following spatial characteristics:

- Immediately adjacent to and upslope from structures forming flow barriers or restrictions to the general trend of within seam flow,
- An adjacent high rate of change of slope (curvature) tending to localised minima where both slope and curvature tends to zero,
- A coincident or immediately adjacent change in aspect from the general aspect trend.

In general, very localised areas featuring all of the above characteristics tended to exhibit higher gas content towards the upper extreme of the sample range. As these gas content observations also approached the sorption capacities displayed on the experimentally derived gas isotherm, it is suggested these areas are at or near saturation for the given seam reservoir pressure.

CONCLUSIONS AND FUTURE DIRECTIONS

Over 2500 gas core samples from two southern Sydney basin mines producing metallurgical coal from the Bulli seam have been analysed in various geospatial context. A robust foundation for the process to obtain, prepare and load the relevant spatial input datasets into a predictive model has been described. Spatial relationships between measured gas content, gas composition, and spatial parameters such as RL, slope, aspect and curvature have been determined. The relevance and importance of determining these relationships at a localised or site-specific, rather than regional level has been demonstrated.

Further development of the predictive model to include material property dimensions, full three-dimensional assessment of proximity to adjacent structures and gas drainage holes will significantly improve model outcomes. This will allow further application of the model to site specific and more complex geology. The results suggest variability and limitations associated with the present traditional approaches to gas emission prediction and design of gas management practices may be addressed using predictions derived from improved spatial datasets, and analysis techniques incorporating fundamental physical and energy related principles. This foundation will allow increasingly complex factors, such as strata material properties, and stress directions and magnitude to be incorporated into predictive models.

The application of physical and spatial techniques described enhances the potential for use of high volume and high resolution real time measurement data in management of gas emission.
risk. By proactively addressing such risks earlier in both the gas and mining life cycle, material reduction of costs and improvement of production and environmental outcomes are more likely to be obtained.

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AN INVESTIGATION OF THE COAL SEAM GAS CONTENT AND COMPOSITION IN SOMA COAL BASIN, TURKEY

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ABSTRACT: The Miocene Soma Basin in Turkey is estimated to contain at least one billion tons of lignite and about half of this reserve is present at depths greater than 600 m. In the Soma Basin, Turkish Coal Enterprises (TKI) has conducted open cut coal mining and underground coal mining activities for several decades in the Northern and Central part of the basin. It is known from the mining operations that the Soma coal basin has considerably gassy coal seams, but until now there isn’t any sufficient scientific and technical research about gas content and composition of coal seams in the basin. Recently, coal exploration activities have been extended to the Southern part of the basin by means of exploratory drilling. In this context, 49 coal core samples were collected and were analysed in terms of gas content and composition. The gas content measurements indicate that as much as 4.2 m³/t coal is present in the coal recovered from 1010.50 to 1010.90 m below the surface. The composition of the gas is dominantly methane with more than 80 %. Considering the chemical composition of the gas and gas indices, the source of the coal gas is biogenic probably generated by bacteria that are introduced to the coal seam by fresh water following mainly the normal faults bordering the graben structure. The possibility of coalbed methane potential of the basin is also investigated with regard to preliminary gas content data.

INTRODUCTION

Methane emission continues to cause serious problems during subsurface coal mining operations. These problems can be described as dangerous methane emissions that result in explosions, coal and gas outbursts, firedamp etc. However, methane can be removed from an underground mine by an efficient mine ventilation design. At high methane concentrations in workplaces, a methane drainage system might be used. Both of these applications can only be predicted by measuring the gas content of a coal seam.

Knowledge of the gas content of a given coal seam is most important for assessing the potential danger of methane emission during mining operations (Yalcin and Durucan, 1991). In order to evaluate the potential gas problems of a new mine or unmined areas of an underground mine, gas content measurements are the most important step for mining operations (Diamond and Schatzel, 1998). During the phase of an underground mine development, gas content measurements can be achieved from surface exploration drillings (Diamond, 1979; Diamond and Schatzel, 1998). An early assessment of the potential for methane emission problems provides the greatest amount of lead time to incorporate longer term gas drainage techniques into the mine development plan (Diamond, 1994). Once mining is underway, gas content testing can be used periodically to assess gas content conditions ahead of mining. Gas content data are also vital for determination of the commercial potential of a field, and core analyses provide that information. Economic production of coalbed methane depends on the amount of gas content. To understand the gas type that releases from the coal seams and neighboring strata, gas composition measurements should be performed.

In the Soma coal basin, coal seams are known as gassy since TKI operated the coal seams for decades. Because of this, the virgin coal seams, where depths are greater than 600 m,
must be measured in terms of gas content before the mining operations. This paper involves
gas composition analysis and a preliminary gas content study of an underground coalmine
that has currently been established in the Eynez part of the Soma coal basin, Manisa, Turkey.
The purpose of the study is to collect sufficient gas content data from surface drillings and to
perform a preliminary gas content assessment. Coalbed methane potential of the area is also
investigated.

STUDY AREA

The Soma Basin has the most significant coal deposits in western Turkey. The total coal
reserves of the Soma Basin are about 719 Mt; about 10.3 Mt of coal are annually produced
by mainly open-pit mines, of which annually 7.7 Mt are used in Soma coal-fired power plants
with 990 MW total installed capacity, whereas 772,325 tonnes are used for domestic heating
and industrial purposes (TKI-ELI 2016). The annual production capacity of the new mine in
the Eynez part of Soma coal basin is planned to be 5 million tons and the infrastructure will be
adequate for 8.5 million tons/year capacity. Production is planned to commence in 2018 for
the underground mine, which will be the deepest lignite mine in Turkey (Mining Turkey, 2016).

In the Soma coal basin, two common formations exist; the Soma and the Denis Formation.
The Soma coal basin extends in a NE–SW direction in an approximately 20-km long and 5-
km wide, fault-controlled basin in western Turkey. The major coal-bearing Soma Formation
started deposition during the Early to Middle Miocene (Seytoglu and Scott, 1991; Inci, 2002;
Karayigit et al., 2017). A generalized stratigraphic illustration of the Soma coal basin and the
location map of the study area are given in Figures 1 and 2 respectively (Karayigit and
Whateley, 1997; Inci, 1998a; Karayigit et al., 2017). Paleozoic and Mesozoic basement rocks
remained under the influence of Alpine Orogeny and as a result of folding and faulting,
especially graben type faultings, they formed basins necessary for the deposition of Miocene
deposits. Neogene is represented by Miocene Basement series (composed of pebblestone,
sandstone and clay, (M1), Lower Lignite series (KM2), Marl series (M2), Limestone series
(M3), Middle Lignite series (KM3) and Pliocene Sandstone-Siltstone-Mottled Clay series (P1),
Upper Lignite series (KP1), Clay-Tuff-Marl series (P2ab), Clay-Sandstone-Pebblestone series
(P2c) and Silicified-Limestone-Tuff series (P3) from bottom to top respectively.

The study area involves coal seams that are independent from each other due to geological
conditions. Eynez part of the Soma coal basin has three coal seams and are named KP1
(upper seam), KM3 (middle seam), and KM2 (main seam). According to mining operations,
only the KM2 seam will be mined after termination of exploration drillings. As it is shown in
Figure 1, the KM2 coal seam was settled on the M1 formation and the thickness of the KM2
seam ranges between 3.5 and 30 meters. The KM2 seam generally shows hard, massive and
bright coal perspective. On the M3 limestone unit, the KM3 seam was settled and it shows a
banded, bright black coal perspective. Average thickness of the KM3 coal series varies
between 1 and 7 m. P1 series are followed by KP1 coal seam that is called the upper seam.
According to Brinkmann et al. (1970), the KP1 coal series make an indicator series, which is
important parameter for geologists. Thickness of the KP1 coal seam varies from 0.5 to 4.5
meters and it doesn't have economical value in terms of the mining industry. Towards
the study area from the TKI mine, coal depth increases as well. Due to past mining
experiences in the mentioned area, it is believed coal seams have regional high gas
concentrations. In accordance with the probability of increasing gas content, it is necessary to
investigate the gas content of the coal seams in this area.
Figure 1: Generalized Stratigraphic illustration of the Soma Basin (from Karayigit et al., 2017).

Figure 2: Location map of the study area in the Soma coal basin.

MATERIALS AND METHODS

Coal gas content analyses for 49 samples from 18 wells and coal gas composition analyses for 6 samples (Table 1) were carried out both in the field and at Istanbul Technical University, Faculty of Mines, Mine Ventilation and Safety Laboratory. Coal seam gas content determinations have been based on following the “USBM Gas Content Direct Method” and
ASTM D7569-10 “Standard Practice for Determination of Gas Content of Coal-Direct Desorption Method”. Gas composition analyses have been performed via Gas Chromatography device. Proximate analyses were performed with ASTM Standards that are very useful for classifying the coal seams in terms of their gassiness and their chemical properties.

**Gas Content Measurements**

Gases associated with coal seams are formed as a result of the coalification process. Coal seams can contain a mixture of gases in which methane makes up 80–90 % (Creedy, 1991) and varies from 0 to 25 m³/t (Noack 1998). Minor amounts of carbon dioxide, nitrogen, hydrogen sulfide, and sulfur dioxide make up the other components of coal seam gases (Flores 1998). It is necessary to determine the coal seam gas content and its gas composition beforehand to protect the mining workplaces and also to determine the gas potential of the mining area. A coal seam gas content named as total gas content can be determined by three components. These components are “Lost Gas”, “Desorbed Gas or Measured Gas” and “Residual Gas” which are determined by the rules of USBM Gas Content Direct Method (Bertard et al., 1970; Diamond and Levine, 1981; Diamond et al., 1986; Diamond and Schatzel, 1998).

### Table 1: Collected coal samples and their properties.

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Coal Seam</th>
<th>Depth (m)</th>
<th>Canister No.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>From</td>
<td>To</td>
</tr>
<tr>
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<td>KM2</td>
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<table>
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Lost gas, which is called Q1, is the important part of the total gas content, which can be estimated only from the initial gas readings of the coal by a plotted graph of desorbed gas. On the other hand, lost gas is the gas that releases from the coal sample during coring until it is sealed into the canister. Coal sample collection and gas desorption tests are performed during the coring operations and five important times have been recorded for the estimation of lost
gas. These lost gas estimation times are; the time of the first penetration of the coal (time coring started), the time that presents the end of coal penetration (time coring ended), after ending the coal penetration the retrieval of core (core-off bottom time), surface arrival time (time core at surface), and the time when the canisters were sealed (time canister closed). These parameters must be recorded on the canister spreadsheet which is indicated by Barker and Dallegge (2005).

Desorbed gas (Q2) can be measured from a coal sample that is sealed in the canister and a graduated cylinder as it is suggested by Diamond and Schatzel, (1998). The initial readings, especially in the first 3 hrs are very important in estimation of lost gas. Desorbed gas readings must be taken every 5, 10 and 15 minutes for the first hour. Then, measurements can continue with increased times of gas readings which is actually depend on the gas desorption rate. Generally, the termination of desorbed gas measurement is when the daily emissions were less than an average of 10 cm³ of gas desorption per day for one week as suggested by Diamond and Levine (1981). Once a coal sample ceases to release more gas effectively, then the coal sample is removed from the canister for determination of Residual gas (Q3) in the grinder. Following the termination of desorption tests, coal sub-samples were taken into grinder for residual gas measurement. Each coal sample were pulverised under 250 microns (60 mesh). This causes opening of the micropores of the coal matrix which includes the gas molecules at the adsorbed state. Finally, total gas content of each coal samples were determined by the given equation (Diamond and Schatzel, 1998);

\[ Q_T = \frac{Q_1 + Q_2}{M_t} + \frac{Q_3}{M_c} \]

In this equation;

- \( M_t \) = coal mass that is collected in desorption canister (g).
- \( M_c \) = coal mass that is used in residual gas analyses (g).

Provided coal cores from the wire-line coring were collected and selected macroscopically and sealed in the airtight and stainless steel desorption canisters. The desorption canisters have 7.0 cm inner diameter and 30 cm length (Figure 3). In the coring operation, the diameter of the drilling core was NQ (inner diameter 4.76 cm) and core was sectioned into the desorption canister as closely as possible to minimize the headspace volume. Desorption canisters were also filled with distilled water to remove headspace calculation errors. Both ambient temperature and barometric pressure were also recorded during each gas measurement. The reservoir temperature was measured at the drilling site and selected as 28°C. All measurements were entered into a canister spreadsheet for determining the coal seam gas content.

![Figure 3: A view of desorption canisters in water at 28°C reservoir temperature.](image)

**Gas Composition Measurements**

In order to determine chemical composition of coal gas, a gas chromatograph is utilized. Since gas contents of lignite seams are commonly low, it is imperative to apply
chromatography techniques. A Gas Chromatography (GC) device can work with gas samples with limited volume. Agilent 7890A model gas chromatography is utilized to analyse chemical compositions of gas. The device includes two detectors at the end of the columns. Flame Ionization Detector (FID) is utilized to detect hydrocarbons in coal gas content and Thermal Conductivity Detector (TCD) is utilized to detect the remaining gas compounds. Air contamination during gas composition measurements is eliminated by normalization calculations, that, equivalent atmospheric nitrogen and carbon dioxide of oxygen is subtracted from the results. For determining the composition of coal seam gas in the Soma coal basin, some coal core samples were taken in canisters and the gas volumes were taken into vacutainers during desorbed gas measurements (Figure 4).

![Figure 4: Gas sampling at the field using vacutainers.](image)

**Proximate Analysis**

Proximate analyses were performed for better understanding of a coal seam gas classification at a mining area. It is also required for a gas assessment because coal type should be determined to classify the seams according to their gassiness. Coal cores were divided into sub-samples pulverized for the residual gas measurements. The testing procedure adopted throughout for proximate analysis conformed to the appropriate ASTM Standard for coal analysis and testing (ASTM D2013/D2013M-12; D3173-11; D3174-12; D3175-11). In summary, this procedure involved the drying of a known mass of coal in an oxygen-free oven at 107°C for a period of one hour. After removal from the oven, and subsequent to the sample being placed in a desiccator, the coal was weighed, and the loss of mass ascribed to inherent moisture. Ash content determination was achieved by combusting the coal until a constant mass was attained in an ash furnace. This was achieved by heating the sample to 500°C for 1 hour before increasing the temperature to 750°C, until combustion was complete. Then, coal samples were stored in the furnace for 2 hours at 750°C. The percentage of ash was calculated from the mass of the residue remaining after incineration. The sample was then heated in a cylindrical silica crucible in a muffle furnace at 900°C for seven minutes. The loss of mass recorded during this process equated to the proportion of volatile matter present in the sample. The amount of fixed carbon was not determined directly, but represented the difference between the sums of all other components.

**RESULTS AND DISCUSSION**

49 coal core samples from 18 different surface exploration drillings were analysed for gas content and gas composition tests. 17 coal samples were taken from KP1 coal seam and 32 coal samples were taken from KM2 coal seam for gas content testing. Gas content (lost, desorbed and residual) and gas the composition of coal samples were determined by measurements in both field and laboratory. The proximate analysis of samples was conducted in the laboratory to classify the coals based on ASTM Standards.
It is known from the basic literature that the gas content of coal seams increase with increase in depth. The gas content, ash content and the moisture content of coal correlated with depth. The relationship between depth and total gas content of coal is shown in Figure 7. Gas content of the coal samples ranged between 0.48 – 4.20 m³/t on as-received basis, 0.90 – 7.26 m³/t on dry and ash free basis. The high values of gas contents on a dry and ash free basis might be caused by high values of ash contents in some samples. Although the correlation of gas content on an as-received basis with depth is low, the increasing trend corresponds with the gas content of the coal seams being increased by the increase of depth. The moisture and ash content of the coal seams on an as-received basis varied between 3.00 – 24.56 %, and 2.61 – 61.91 % respectively. Volatile matter of the coal seams are varied between 4.06 – 47.07 % on an as-received basis and 40.74 – 88.54 % on a dry and ash free basis. In this study, moisture content (Figure 8) and the ash content (Figure 9) of the coal seams decreased with increasing depth. According to ASTM standard, coal samples used in this investigation are classified as lignite but much closer to “sub-bituminous coal” due to volatile matter of 80 % of the coal samples being above 50 %. This is caused by high values of ash and moisture content that affects calculation of the volatile matter of coals.

Figure 7: Relationship between coal seam gas content and its change with depth.

Figure 8: Change of moisture content in coal with increase of depth.
The results of gas composition of coal are given in Table 4. To perform the gas composition tests, 6 coal samples were taken from the KM2 coal seam due to its economical importance. The results show that methane is the dominating compound in the gas content with relatively lesser variation in other gases. Apart from other factors, gas composition of the seam is favorable for CBM production. The ratio of C1 / (C1+..+C5) is 99.8 % in average with negligible variation suggesting dry CBM (Li, et al., 2015).

Table 4: Results of gas composition measurements of the KM2 coal seam.

<table>
<thead>
<tr>
<th>Coal Seam</th>
<th>Canister No.</th>
<th>Depth (m)</th>
<th>CO2 (%)</th>
<th>He (ppm)</th>
<th>H2 (%)</th>
<th>N2 (%)</th>
<th>CO (%)</th>
<th>CH4 (%)</th>
<th>C2H6 (%)</th>
<th>C2H4 (%)</th>
<th>C3H6 (%)</th>
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<td>24.7</td>
<td>0</td>
<td>21.25</td>
<td>9</td>
<td>0.076</td>
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<td>2.323</td>
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</table>

Thakur (2011) has classified the coal seams that can be divided into three categories according to their gassiness and depth (Table 5). According to classification of Thakur (2011); in the Eynez part of the Soma coal basin, coal seams are classified as “Mildly Gassy” and “Moderately Gassy” in accordance with the average gas content of 1.67 m³/t and the maximum gas content of 4.20 m³/t. Furthermore, the maximum gas content value has an evidential point that it is provided from the depths between 1010.50 – 1010.90 meters and from PF-28 exploration well.
Table 5: Gassiness classification of the coal seams.

<table>
<thead>
<tr>
<th>Category of Mine</th>
<th>Depth (m)</th>
<th>Gas Content (m$^3$/t)</th>
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</thead>
<tbody>
<tr>
<td>Mildly gassy</td>
<td>≤ 200</td>
<td>&lt; 3</td>
</tr>
<tr>
<td>Moderately gassy</td>
<td>200 to 500</td>
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<tr>
<td>Very gassy</td>
<td>&gt; 500</td>
<td>10 – 25</td>
</tr>
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</table>

Moreover, gas content values were considered while calculating the in-situ gas potential of the mining area. According to the field observation and investigation that in situ gas potential for CBM production has been estimated as approximately above the 300 million m$^3$, which corresponds with a 150 million m$^3$ coal reserve.

ACKNOWLEDGEMENTS

Grateful thanks to Professor Naj Aziz for his helpful and fruitful support and giving us the chance for present this paper. Furthermore, the paper benefited from valuable support by y. The authors also would like to express their gratitude to Polyak Eynez Mining Corporation in Soma, Turkey for providing coal samples and performing the surface exploration drillings in the Soma Eynez coal basin.

REFERENCES

POTENTIAL TUBE BUNDLE IMPROVEMENTS
Larry Ryan¹, Martin Watkinson²

ABSTRACT: The gas monitoring system in an underground coal mine is an integral part in creating a safe work environment for the coal mine workers. The gas monitoring system provides feedback on the effectiveness of the ventilation system, gas drainage system, seal integrity of goafs and gas make on the active face/development panel.

The aim of this paper is to provide a number of examples, where possible improvements can be made to Tube Bundle monitoring systems. Two of the Tube Bundle systems biggest limitations are slow response times to changing gas conditions underground and the high cost of tube installation underground. This paper will discuss two possible step change improvements with the aim to address these limitations to make the Tube Bundle system more responsive and provide greater coverage underground for lower cost outlay. This paper will also explore the use of automation to reduce the cost of Tube Bundle system maintenance while improving reliability.

DECREASE PURGE DELAY TIME – TUBE BUNDLE SHED

A typical Tube Bundle system in an underground coal mine consists of 20 – 40 tubes, each approximately 5 km long, sampling various areas underground on a continuous basis with each tube being either 12.5 mm (½") OD (Outside Diameter) or 15.9 mm (5/8") OD. Most mines have moved to 15.9 mm (5/8") tube as this allows for longer runs of tube underground, up to 12 km due to the lower resistance in the tube.

Unfortunately, the longer the tube the greater the vacuum required to drawn the same gas sample flow (litres/minute) to the surface. In addition, the longer the tube, the further the gas sample, from underground, needs to be transported to reach the surface. Hence a longer time is required to get the sample from underground to the surface for a long tube vs short tube. An analogy for a simple Tube Bundle system would be sucking air through a straw. If you had a standard straw, then it is quite easy to draw some air through the straw by sucking on it (applying a vacuum). However, if the straw is made twice as long, it becomes more difficult to draw air through the straw. If the long straw, above, is changed to a thick straw with a larger Cross Sectional Area (CSA) then the resistance is reduced and it is easier to draw air through the straw again.

Hence to combat the greatest liability of a Tube Bundle system, the time delay from sample collection underground to sample analysis in the Tube Bundle shed, the system needs to increase the sample flow. More sample flow can be brought to the surface faster by either increasing the vacuum applied to the tube or lowering the resistance of the tube. Using the straw analogy, more sample flow can be achieved by either sucking harder or using a thicker straw with a larger CSA.

The mining industry has investigated these two approaches in the past and has opted to lower the tube resistance by using a tube with larger CSA, namely moving to 15.9 mm (5/8") tube instead of 12.5(½") tube. If the tube vacuum were increased, then the Tube Bundle system maintenance/installation would be more onerous as every joint/connection would need to be of a higher standard/quality in order to maintain the higher vacuum. In addition, a higher vacuum Tube Bundle system would be more susceptible to gas leaks.

It has been identified that 12.5 mm (½") tube is actually more robust and as a result is more likely to survive an explosion underground (Queensland Mines Rescue Service et al, 2015). The survivability of the Tube Bundle system needs to be forefront as post explosion, all power underground is likely to be lost (the Realtime system, if it survived, will be running on Uninterruptable Power Supply (UPS) with limited uptime) and hence the Tube Bundle system

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will form part of the re-entry strategy. Hence, ideally the Tube Bundle system should use 12.5 mm (½") tube from a survivability perspective while a 15.9 mm (5/8") tube is better from a purge delay perspective.

For tube runs up to approximately 6 km, 12.5 mm (½") tube is recommended while over 6 km, 15.9 mm (5/8") tube is better from a purge delay perspective. As the purge delay is the biggest disadvantage of a Tube Bundle system, the correct tube size selection and installation are paramount for a minimal purge delay time. However, a Tube Bundle system has more components than simply the underground tube and there is reason to believe that the infrastructure inside the Tube Bundle shed itself may contribute significantly to the purge delay time as shown below in Figure 1.

![Figure 1: Tube Flow Path](image)

Once the tube, from underground, enters the Tube Bundle shed, the tube size typically drops to 6.25 mm (¼"). The CSA of a 6.25 mm (¼") tube is much smaller than 12.5 mm (½") or 15.9 mm (5/8") tube and hence the resistance will be much higher. As the tubes total resistance is the sum of all the various parts, the 6.25 mm (¼") tube inside the Tube Bundle shed could add considerably to the total resistance of the tube run. In addition, the tube typically goes through a flame arrester, water traps, filters, a check valve and other various valves before finally being drawn on and exhausted by the purge pump. Each element in the tube flow path will add to the tube’s total resistance and the higher the resistance, the lower the sample flow, hence the longer the purge delay.

In order to bring a sample, from the end of line filter to the Tube Bundle shed, the volume of the 12.5 mm (½") or 15.9 mm (5/8") tube needs to be drawn out of the tube or purged. As the tube could be 12 km in length, the volume of gas that needs to be evacuated is quite large and any additional resistance by the various Tube Bundle shed components will only add to the tube purge time.

As stated above, the total resistance of the Tube Bundle purging flow path is the sum of the resistance of the tube underground and the resistance of the 6.25 mm (¼") tube/components inside the Tube Bundle shed.

The total resistance of all the components inside the Tube Bundle could be reduced by using the following:

1. 12.5 mm (½") or larger tube inside the Tube Bundle shed.
2. 12.5 mm (½") or larger valves inside the Tube Bundle shed.
3. Lower resistance flame arrestors
4. Lower resistance filters
A flame arrestor, which has a lower resistance, could be of the same conventional design but have a wider CSA so the resistance to flow is reduced. Alternatively, a new large CSA, low resistance flame arrestor may need to be designed for this application.

It is proposed that if the resistance of the tube/components inside the Tube Bundle could be reduced then the purge delay times will be reduced. A smaller purge time would lead to the Tube Bundle system being more responsive to changing underground gaseous conditions and hence safer outcomes for the mine workers. The sourcing of parts, construction and testing of this particular potential improvement is beyond the scope of the current paper unfortunately.

**AUTOMATION OF TUBE BUNDLE SYSTEMS**

Tube Bundle systems have a high initial cost and maintenance requirements. In order to reduce the total cost of ownership, this paper will propose five ways that automation can either extend the functionality of the Tube Bundle system or reduce the labour required for maintenance.

**Tube Monitoring Expansion**

The high cost of installing a Tube Bundle system is made up of two major hardware components:

5. Surface infrastructure – Tube Bundle shed containing the various pumps, valves and analysers,

A significant proportion of the cost of installing a Tube Bundle system is for the multicore and single core tube run underground. Once all the tubes have been run underground to meet the maximum capacity of the Tube Bundle system, expansion of the Tube Bundle system or the number of monitoring locations is difficult.

Each tube being monitored by the Tube Bundle system needs to run from the Tube Bundle shed, at the surface, to the monitoring location, underground several kilometres away, as one continuous tube. Hence, each tube needs to run down a bore hole or drift (generally as multicore) to a location central to the area to be monitored and at this point, individual single core tubes are run to each discrete monitoring location. Depending on the mine and the area to be monitored, the multicore run could be two or more kilometres in length before the individual tubes branch out toward their particular monitoring location.

The multicore tube run that is common to all the tubes destined for monitoring in a particular area represents an opportunity to use automation underground via Pneumatic (compressed air) to either:

7. Expand the number of locations that can be monitored
8. Decrease the purge delay times

If the control system, in the Tube Bundle shed, was upgraded to control a number of pneumatic lines going underground as well as the standard valves, the opportunities outlined above could be realised. Instead of just using each individual tube in the multicore to monitor just one location underground, it would be possible to have a pneumatic valve switch between two or more tubes. The basic principle is to use a single tube, in the multicore bundle, to transport the gas sample from more than one location underground to the surface as shown below in Figure 2.
However, in order to achieve this outcome, the following requirements will need to be met:

9. Each tube connected to a pneumatic valve needs to be purged long enough to completely flush the previous tubes’ gas sample from the common multicore tube. Using the new sample to flush out the previous tube’s sample is the same principle that is used in the Tube Bundle shed to flush the analyser’s flow path.

10. The control of the tube sampling order needs to be tightly controlled to ensure that the common multicore tube is flushed long enough to remove all residual gas from the previous tube.

11. The tube delay time from the pneumatic valve to the surface needs to be found to assist in the calculation of the time required to flush the old tube sample out. For example, it may be found that the common multicore tube needs to be flushed three times to remove all evidence of the previous sample (the time or flushing cycle involved to flush the previous sample, whether it is dirty air, high CH4 or high CO2 is beyond the scope of the current paper unfortunately).

12. The maintenance of the Tube Bundle system in relation to the common area pneumatic tube panel would need to be of a high standard.

13. Tube integrity testing would need to be undertaken to ensure that the system operated correctly and that the flushing time of the common multicore tube is long enough.

14. Water cannot be allowed to pool in the common multicore tubes as high CO2 samples will dissolve in water and then degas into another sample.

The pneumatic air lines would need the run to the central location and probably the best way to achieve this would be by having the air lines as part of the multicore bundle. Hence, the multicore bundle is run to the central location where there is a panel, similar to a marshalling panel, containing the pneumatic valves to control the switching of the tubes. Using the
pneumatic valves, two or more tubes are to connect to a single multicore tube that then transports the gas sample to the surface.

**Decrease Purge Delay Time – Underground Tubes**

Another, slightly more complex variation of the tube monitoring expansion principle outlined above, is to use two or more common multicore tubes in parallel to reduce the purge delay as shown in Figure 3. Due to the complexity of managing the pneumatic valves and flushing times, the usage of parallel common multicore tubes will probably be limited to longer tube runs (the time or flushing cycle involved to flush the previous sample, whether it is dirty air, high CH₄ or high CO₂, is beyond the scope of the current paper unfortunately).

![Figure 3: Parallel Flow Path](image)

**Automated Blow Back**

While not common, there are Tube Bundle systems in use that incorporate automated blowback functionality. Automated blow back provides a scheduled, low maintenance method of ensuring that minimal water is collected in the tube as this tends for increase the purge delay times. The other advantage of automated blow back over manual blow back is there is no need to disconnect and then reconnect the tubing which has the potential to introducing sample leaks.

As the blow back functionality uses high pressure air/N₂ to blow the water out of the Tube Bundle system, there is potential that high pressure air/N₂ could damage other sensitive equipment (for example, the gas analyser). Hence, the design of the automated blow back system needs to ensure that high pressure air/N₂ cannot be released in the same flow path as the gas analyser and other pressure sensitive items.

**Automated Tube Integrity Testing**

The monthly tube integrity testing process is a high maintenance task that can take a long time to complete (especially if sample leaks are found). Currently, an automated tube
integrity system has been developed where $N_2$ is injected down the tube and then the $N_2$ is drawn back to the surface via the Tube Bundle system. If the gas sample drawn back is purely $N_2$ then the tube doesn’t have any leaks. The automated tube integrity system cannot confirm that the tube located at a particular location underground has not been swapped with another. Tube identification requires a known gas to be drawn at end of the tube and then a matching gas sample found at the surface to positively identify the tube.

**Automated Analyser Calibration**

Various Tube Bundle systems incorporate automatic analyser calibration to minimise the maintenance requirements. Generally, in order to calibrate the analyser automatically, the calibration cylinders need to be left on which could pose a health and safety hazard. In addition, some of the calibration gases are special mixtures and could take up to six weeks to be made. If the calibration gas line had a small leak, leaving the calibration gas cylinder on could cause the cylinder to be empty when required. In the worst case, the automatic calibration may be attempted using an empty cylinder which would result in a bad calibration and if no spare cylinders were available, leading to a period of time where the analysis of that particular gas would be unreliable.

There are analysers on the market that insert a glass gas calibration reference cell into the light path of the analyser; hence an internal reference is used for the calibration rather than an external gas source. The use of an analyser with this internal calibration capability provides an opportunity to capture the benefits of automation without introducing potential gas hazards.

**Automation Safe Guards**

While the automation of routine maintenance for a Tube Bundle system certainly has benefits, there needs to be a series of checks and balances to ensure that the task/maintenance is being carried out correctly and there are no unintended side effects.

As mentioned above, automated blow back has the capability to destroy various Tube Bundle components (for example, the gas analyser), if there is an uncontrolled release of gas pressure. Automated calibration has worked so well, in the past that no site inspections/maintenance of the Tube Bundle system was being undertaken. Unfortunately, when no regular maintenance of the Tube Bundle system is being performed, small maintenance issues were left unresolved which has then led to catastrophic failures (for example, water in the gas analyser). There is also a concern that the automation of Tube Bundle systems will lead to the mine’s personnel having diminished knowledge as they lose their familiarity with the Tube Bundle system. In addition, as the more routine tasks are automated (for example blow back, tube integrity checks and analyser calibration) the support/breakdown issues remaining will be more challenging for the personnel on site to resolve.

Maintaining a close working relationship with the original equipment manufacturer (OEM) would be of assistance with support/breakdown/parts replacement in the future. In addition, if the six monthly NATA calibrations are performed by an experienced service person, maintenance can be undertaken and any issues identified can be rectified at the same time.

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IMPROVING RESPIRABLE COAL DUST EXPOSURE MONITORING AND CONTROL

David Cliff¹, Nikky LaBranche², Mark Shepherd³, Fritz Djukic⁴

ABSTRACT: This paper will present the progress results of this research project commissioned by ACARP to establish the state of the art with respect to:

- Current dust suppression and prevention controls and their effectiveness,
- The basis for setting an exposure standard for respirable coal dust,
- The current trends in exposure data and any underlying points of concern or interest,
- The capacity to monitor respirable coal dust in real time – what techniques are available and what are their limitations,
- Current research into reducing exposure to respirable coal dust,
- Future directions for research into better control of respirable coal dust exposure.

INTRODUCTION

Since the first case of Coal Workers Pneumoconiosis (CW) was reported in Queensland in 2015 in over twenty years there have now been a total of 56 confirmed cases of Coal Mine Dust Lung Diseases (CMDLD) among current and former Queensland mine workers (DNRM, 2017a). Coal mine dust lung diseases are caused by long-term exposure to high concentrations of respirable dust, generated during mining and quarrying activities. CMDLD include a range of occupational lung conditions including:

- Asbestosis;
- Coal Workers’ Pneumoconiosis (CWP);
- Chronic obstructive pulmonary disease;
- Silicosis.

Two parliamentary inquiries - one Federal and one State, have been convened. Because of these there will be major changes in the way mines and the State Government approach the management of worker exposure to respirable coalmine dust. Much effort has already been devoted to improving the coal worker health scheme to remove the problems in diagnosis that were found by the Monash Review (Sim et al, 2016), including coverage for retired workers. It has also generated a number of questions relating to monitoring and control of the coal dust exposure. In recognition of this ACARP established a project C 26048, this research seeks to answer or at least provide more detail on and allow for better definition of a number of questions. These questions include:

- Do we need better dust control systems?
- Are current monitoring techniques adequate?
- Do we need a lower coal dust exposure standard?
- Is coal dust the problem?
- What are the gaps in our knowledge of respirable coal dust exposure?

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CURRENT RESPIRABLE COAL DUST EXPOSURE

In response to the CWP situation DNRM has issued two recognised standards – one on monitoring of dust (Recognised Standard14, DNRM, 2017b) aimed at improving the quantity and quality of the respirable coal dust monitoring and the other on dust control techniques (Recognised Standard 15, DNRM, 2017c).

In 2015, the DNRM initiated a program to collect all the respirable coal dust monitoring information from underground mines in Queensland. The DNRM currently reports this data on a quarterly basis. Figure 1 shows the trends in exposure data since 2014 for the Longwall Similar Exposure Group (SEG) by de-identified mines. Following an increase in the focus on dust control since 2015 the average exposure has fallen from close to the regulatory exposure standard of 3 mg/m$^3$ to around 1 mg/m$^3$. This would suggest that if applied the current technology mines have is capable of bringing the exposure down to well below regulatory limits.

In conjunction with DNRM this research project staged two workshops in Central Queensland where mines were able to present their current respirable dust control strategies. They demonstrated that major improvements in reducing respirable dust exposure have been achieved through:

- Automation – removal of worker from the active longwall face during mining;
- Greater use of water sprays to suppress the dust – not just on the shearer but also on the shields, rear walkways and AFC;
- Dust maps have been developed identifying areas of high potential exposure so that workers can avoid them – utilising real time dust monitors;
- Experimenting with different types of water sprays and the use of surfactants and foams;
- Enclosure of crushers and transfer points.

This improved application of controls has been supported by greatly increased personal exposure monitoring utilising both whole of shift gravimetric analysis and real time monitoring.

In NSW there is a centralised monitoring regime in place implemented by Coal Services under orders 40 and 42. In addition, a tripartite standing dust committee meets quarterly to review the monitoring results. The results for 2015-2016 are displayed in Figure 2. Only 0.7% of samples collected in 2016 in NSW Longwall mines exceeded the NSW exposure...
standard of 2.5 mg/m$^3$. There were higher levels of exceedances for respirable quartz and inhalable dust.

Figure 2: NSW airborne dust exposure exceedance (Coal Services, 2017)

RESPIRABLE COAL DUST MONITORING

Traditionally monitoring of worker exposure to coal dust has been undertaken using cyclone elutriators for gravimetric analysis of a worker exposure over an entire shift.

This technique is designed for measuring compliance with the exposure standard and does not permit identification of individual dust sources nor whether controls are effective. Control effectiveness is best measured using real time devices that can quickly pick up changes in operating conditions. Real time respirable dust monitors have existed for more than twenty years.

The current research project is carrying out trials to compare a range of real time and near real time monitoring devices for suitability. No one device meets all the required criteria. Ideally a real time dust monitoring device would be:

- Intrinsically safe
- Light weight and comfortable to wear
- Capable of running for at least 12 hours
- Report actual respirable coal mine dust concentration in real time, unaffected by moisture.

The PDM3700 using tapered element oscillating microbalance (TEOM) technology is approved for use in the USA by MSHA and has been designated as the default monitoring device there (Volkwein, 2017). It is not certified for unrestricted use here but can be used as uncertified portable electrical equipment. It has been widely applied over the past twelve months in Australia and has generated much useful data. Unlike other real time devices it actually undertakes gravimetric analysis in near real time – providing the user with a running 15 minute average concentration of respirable coal dust. Apart from the difficulties in certification the other minor issues with its operation that have been identified relate to its bulk and cost.

The majority of other real time devices utilise some form of light scattering technology to determine the dust concentration. This requires the calibration of the device with the dust of interest to get an accurate response. In addition water droplets such as are used in suppression systems would be detected by these devices, causing inaccurate readings. The PDM 3700 utilises a heated inlet system to prevent this problem.

All these devices measure the total respirable dust in the air. Currently there is no device that looks at the speciation of the dust in real time. Traditional analysis for silica content requires
the gravimetric filter to be processed in a laboratory. Recently NIOSH in the USA have developed a technique that allows the filters to be analysed on the mine site (Miller, et al, (2017). This in turn enables feedback to the mine and miners for the next shift rather than having wait weeks for the laboratory report. This immediacy of response greatly improves the mines capacity to identify potential problems and implement solutions before the exposure becomes excessive.

**OCCUPATIONAL EXPOSURE STANDARD**

Perhaps the biggest question raised by the research so far is what is the cause of the CWP and thus what should be the exposure standard(s). Currently medical experts are careful to talk in terms of coalmine dust rather than coal dust. This is because there may be components of the dust other than coal that could be causing the CMDLD, such as silica, and bioavailable iron. Recent research from the USA, where there are currently over 1000 new cases of CWP reported annually, indicate the potential contribution of non coal dust to the exposure (figure 3). Many of these components have much lower occupational exposure limits than coal dust (silica is currently 0.1 mg/m$^3$). It is possible that the cases of CWP reported in Queensland are not due to coal dust but silica or other components. It is important to identify the source so that the correct pathogen is controlled and exposure standards are not unnecessarily reduced, which would create an associated financial burden on mines.

![Figure 3: Respirable coal dust composition in East USA (Johann-Essex et al, 2017)](image)

**RESEARCH NEEDS**

ACARP has already commissioned new research to begin this year (2018) investigating the potential causes of Queensland CMDLD in terms of the active species responsible. This knowledge is vital before attempting to set any new occupational exposure standards especially if the exposure standard applies not only to CWP prevention but also all other CMDLD.

Work has begun to undertake a detailed analysis of the existing exposure data from Queensland and NSW to see if there are any insights that the data can offer in terms of
control effectiveness and the risk profiles for different worker groups.

Recent cases of CWP have emerged in the open cut coal industry and concern is now also spreading to the quarrying sector, tunnelling and metalliferous sectors, which do not have the centralised compulsory medical system that exists for underground coal nor, in many cases, the monitoring regimes.

There is a need to develop real time fixed point monitoring devices that are capable of monitoring the effectiveness of controls and reporting when the controls are not effective. This would require the issues relating to water droplets to be resolved as well as making devices robust enough to survive the underground coal working environment.

A number of the controls currently applied need to be optimised for effectiveness and tailored to the particular situation where they are applied i.e. water spray droplet size and energy in comparison to the target dust particles.

Automation and segregation of the worker from the dusty environment have delivered major benefits but there is still work to be done in ensuring high levels of availability of the automation systems while delivering the production targets.

CONCLUSIONS

- The research project is currently undertaking factorial analysis of respirable dust exposure data from NSW and Queensland in an attempt to gain better insights into the nature of the problem and potential control strategies.
- The literature review delving into monitoring techniques, control technology and the epidemiology of coal mine dust exposure is still in progress.
- Trials comparing various real time devices are currently in progress.
- A final report is due in May of 2018 and the findings will be publicised at the NSW and Queensland Mining Industry Safety and Health conferences.

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EVALUATION OF GRAVIMETRIC SAMPLERS AND PROPOSAL FOR THE USE OF A HARMONISED PERFORMANCE BASED DUST SAMPLER FOR EXPOSURE ASSESSMENT

Bharath Belle

ABSTRACT: The period of the last three years brought about alarming news of re-identification of Coal Worker’s Pneumoconiosis (CWP) or “black lung” in Australia after reports nearly being absent for over five decades. In South Africa, the CWP statistics are unverifiable, but certainly they have not been eliminated. These events have re-kindled the need for better understanding of dust monitoring, performance of sampling devices and compliance determination. Over the last half century, gravimetric sampling has been the fundamental means for dust exposure monitoring using recognised respirable size-selective standards. In both South Africa and Australia, gravimetric sampling technique in coal mines has been followed since 1988 and 1983 respectively using samplers of original Higgins-Dewell (HD) type designs. With an aspirational mine dust exposure limit of 1.5 mg/m³ after the revision of US dust standard, it is equally important to understand the sampling tools used for exposure monitoring. This paper provides the evaluation results of currently used South African and Australian gravimetric samplers compared against the original UK SIMPEDS ‘true reference’ sampler. The results consistently suggested that the South African and Australian cyclones do not conform to the required BMRC or ISO 1995 curve. The results show that the currently used SA and Australian instruments showed a D₅₀ sampling bias as high as 59% and 47% respectively against the size-selective curve. Similarly, when tested under the controlled laboratory coal dust test conditions, the measured levels by South African, Australian and UK standard SIMPEDS sampler were 8.4 mg/m³, 9.8 m/m³ and 6.7 mg/m³ respectively, aligned with the sampling bias. The differences can in part be attributed to the ‘un-auditable’ inherent design and manufacturing quality, or unverifiable data on the size-selective sampling curve. This finding has significant implications towards exposure data collected over the last 25 years and their subsequent use in the arrival of the dose-response curves. Therefore, it is strongly recommended that the harmonised use of ‘true reference’ SIMPEDS cyclone that meets the ISO (1995) criteria uniformly across the industry would benefit the exposure assessment and compliance determination as practiced in the USA.

INTRODUCTION

Respirable dust sampling is pivotal in estimating the ‘dose’ of individual worker exposure to dust and in deriving quantitative respiratory disease risks in epidemiological studies. Based on the past epidemiological knowledge (Orenstein, 1960), it has been established that the respirable dust particle size distribution is critical due to its potential health effects and the need to quantify the risks. Respirable dust refers to particles that settle deep within the lungs and that are not ejected by exhauling, coughing, or expulsion by mucus. Since these particles are not collected with 100% efficiency by the lungs, respirable dust is defined in terms of size-selective sampling efficiency curves. This had led to internationally recognised respirable size-selective sampling (Orenstein, 1960) widely known as the British Medical Research Council (BMRC) definition of the respirable dust fraction or Johannesburg curve with a median aerodynamic diameter of 5 μm collected with a 50 % efficiency (D₅₀). In reality these size-selective curves represent lung penetration of dust particles that dust sampling instruments attempt to replicate. The International Standards Organization (ISO) in 1995 recommended that the definition of respirable dust follow the theoretical convention described

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by Soderholm (1989, 1991) with a $D_{50}$ of 4 μm. An international collaboration (ACGIH, 1985, ACGIH 1999, ISO 1995, CEN, 1993) for sampling harmonisation has led to the agreement on the definitions of health-related aerosol fractions in the workplace, defined as the inhalable, thoracic and respirable curve. Figure 1 summarises the BMRC and ISO size-selective curves for dust sampling in mines (NIOSH, 1995; ISO1995). The new respirable size-selective curve is different from previous definitions used in the United States, South Africa, Australia and Europe and truly represents an international harmonization of the definition of respirable dust.

Figure 1: Respirable dust size-selective sampling curves.

Therefore, for any personal exposure monitoring, the chosen respirable dust sampling device should achieve the theoretical sampling definition criterion as closely as possible to minimize bias using the $D_{50}$ performance criteria at the recommended flow rates. Due to the complex nature of sampler performance evaluations and their differences, regulatory bodies have dealt with this aspect by decreeing one specific sampling device, (i.e., MRE, Dorr-Oliver, HD), as the reference sampler of choice. What is important herein is whatever sampler is used for exposure measurement; they are to be referenced to epidemiological health effects data to derive any meaningful benefits.

Formerly, sampling conventions corresponded more to some device than to health related issues. E.g., BMRC respirable aerosol convention adopted in 1959 at the Johannesburg Pneumoconiosis Conference (Orenstein, 1960) fitted the efficiency of the MRE 113A horizontal elutriator. In addition, a dust sample collected by some sampler used in a country was declared to be “respirable aerosol fraction” and thus many “reference samplers” found in the literatures. With the ISO (1995) harmonisation curve resulted in the standardisation of health-related aerosol fractions independently from the samplers used but the standardised “size-selective specification” to be conformed by any compliance sampler. As a result, there were modifications and operation of the samplers required samplers to be tested in ideal conditions to yield their sampling efficiency curve and their performance expressed by bias maps. While there may be differing views on the choice of sampler to be used in the industry, the use of $D_{50}$ as selection criteria is the only widely used and accepted criteria for dust sampler selection, in conjunction with the comparative laboratory concentration tests under controlled calm air conditions.

BASICS OF PERSONAL DUST SAMPLERS

The primary purpose of personal respirable dust sampling is to characterize (with regard to mass and size) the quality of the ambient air to evaluate a miner’s dust exposure. The mass of respirable dust inhaled can be determined by sampling. The measurement of dust in mines is usually carried out using various gravimetric sampling instruments. For personal coal mine dust sampling, the dust sampler or cyclone is normally mounted on the upper chest, close to the collarbone within the breathing zone (HSE, 2000). The breathing zone is the space around the worker’s face from where the breath is taken, and is generally accepted to extend no more than 30 cm from the mouth. Gravimetric dust monitoring involves sampling a known volume of ambient air through a filter. The filter is weighed before and after exposure to
determine the mass of particles. The collected dust sample is expressed as mass of dust (mg) per cubic meter (m$^3$) of air.

With acceptance of defined gravimetric based size-selective sampling, various types of dust samplers called ‘cyclones’ were developed and used in mines worldwide since the 1960s (NIOSH, 1995). Cyclones are named for the rotation of air within its chamber that separates and selects dust particles of interest from ambient airborne dust. The cyclone functions on the basis of a centrifugal force principle, i.e., the rapid circulation of sampled air separate particles according to their equivalent aerodynamic diameter.

In a cyclone, non-respirable particles are forced to the periphery of the airstream and collected in a grit pot, while the specified particles remain in the centre of the air stream and are deposited onto a pre-weighted filter medium. The size fractions sampled are very sensitive to the type of cyclone used and variations in flow rate. Various commercially available cyclones can approximate specified size-selective curves when operated at certain flow rate. Any minor deviation from the recommended flow rate would lead to differences in measured dust results. For example, a mere change in flow rate of HD type cyclone from 1.9 Lpm to 2.2 Lpm can result in differences of up to 20% in measured dust values (Kenny, Bristow and Ogden, 1996, Belle, 2004). Both South Africa and Australia have adopted the new size-selective curves with a change in sampler flow rates from 1.9 Lpm to 2.2 Lpm. Therefore, there is a need for amendment to the exposure limits to incorporate the measurement differences due to the change in sampling flow rates.

With the advent of the internationally accepted respirable size-selective curves, research studies have compared various dust samplers available for use in mines. What is obvious from the various studies (Liden and Kenny, 1991, Kenny and Gussman, 1997, Gudmundsson and Liden, 1998, Görner, et al., 2001) is that there are significant differences in measured dust levels from different samplers measuring the same aerosol. The reasons affecting the performance of these different dust samplers can be attributed to inherent cyclone design, air velocity, and direction of airflow, humidity, sampler inlet size, geometry, orientation, aerosol particle size, aerosol density differences, electrical charge, particle bounce properties, and conductive properties of cyclones. Globally, over the last 6 decades, various size-selective conventions have been used, as well as various types of personal gravimetric samplers being used by mines. Until recently (Feb 2016, in the USA, the Dorr-Oliver 10 mm nylon cyclone (Jacobsen and Lamonica, 1969, Lippman and Harris, 1962, Caplan, et al., 1977) was the widely used sampler operated at 2.0 Lpm across the entire U.S. coal mining industry. On the other hand, most of the European countries (including UK) use the HD type cyclone (Higgins and Dewell, 1967; Harris and Maguire, 1968; Maguire et al., 1973; Gwatkin and Ogden, 1979; Ogden, et al., 1983; Blackford et al., 1985, Gudmundsson and Liden, 1998). The latest real-time Continuous Personal Dust Monitor (CPDM), PDM3700 uses a HD cyclone operated at 2.2 Lpm and manufactured by MESA Laboratories (USA).

In a review of respirable dust samplers used in mines globally, it is noted that the UK HD plastic cyclone or also called as UK SIMPEDS (Safety in Mines Personal Equipment for Dust Sampling) is used as a reference sampler operating at a flow rate of 2.2 Lpm which has been characterized previously by Maynard and Kenny (1995). The SIMPEDS or Casella cyclone sampler of the generic HD type is recommended for use in the UK for optimal agreement with the respirable convention. Currently, these HD cyclones are referred to by commercial names such as Casella, SKC, BGI, MESA. For all cyclone performance evaluation purposes, HSE uses Casella SIMPEDS plastic sampler as a ‘true reference’ sampler. Some of these HD type cyclones are metal as well as plastic type. It is possible that different laboratories recommend different flow rates for the same cyclone.

Gudmundsson and Liden (1998) investigated various cyclone models in laboratory studies at a flow rate of 2.1 Lpm and observed that $D_{50}$, increased with increasing inner diameter of the vortex tube or surface properties of cyclone material. For example, what this would mean is that Supplier D HD cyclone vortex tube with an inner diameter of 3.12 mm would result in higher $D_{50}$ of 5.32 microns than the Supplier A HD plastic cyclone vortex tube with an inner diameter of 3.02 mm $D_{50}$ of 4.54 microns, a difference in $D_{50}$ of 0.7 microns. The laboratory results and a study by Liden (1993) provide the explanation on the differences (of up to twice
as large) increased measured dust concentrations by supplier D cyclones when compared with the Supplier A metal cyclones. It is certain that manufacturer modifications such as blacking, tapering of the vortex tube inlet, and gasket type do influence the cyclone penetration curves.

**History of South African gravimetric samplers**

The original Department of Mineral and Energy Affairs (DME) document (DME, 1988) titled “Guidelines for the Gravimetric Sampling of Respirable Airborne Dust Concentrations in Coal Mines” for risk assessment in terms of the occupational diseases in mines and works Act (1973) do not refer to specific dust sampler for use in South African mines. However, a note on the instrument acceptable as gravimetric samplers (Grabe, 1988) documents a few samplers and were required to meet the following criteria:

- The particle size distribution of the dust on the filter in the test instrument must comply with the ‘Johannesburg Curve’ for respirable dust, i.e., particle aerodynamic diameter of less than 7 microns.
- The coefficient of correlation must be 0.9 for the linear regression line against MRE 113A gravimetric dust sampler.
- The standard error of estimate must not exceed 10% of the mean sample mass.
- A calibration curve is required for deviations of approximately greater than 10% from the reference curve.

However, the approved sampling cyclones suggested during the 1980s were SKC cyclone, Casella cyclone with relevant filters and sampling pumps to be operated at 1.9 Lpm. There has also been a reference to a Dorr-Oliver cyclone used in conjunction with Chamber of Mines South Africa (COMSA) inhalable dust sampler used in gold mines that were initially operated at 1.0 Lpm and then changed to be operated at 1.85 Lpm (Schroder, 1982) or rounded off to 1.9 Lpm to align with the UK SIMPDES sampler. Another technical note (Lamprecht and Rowe, 1991), documents the use of Gilian GX-37 cyclone, GX-35 cyclone and the Gilian GX-R25 mm cyclone operated at 1.9 Lpm for use in South African mines. However, the evaluations were done merely on mass concentration comparison basis (< 5 % measured difference) and no information on size-selective curves was available.

Although, original gravimetric sampler lists included various traditional cyclone manufacturer trade name/s such as Casella and SKC, their use at mines disappeared from the exposure monitoring regime completely. Currently, in South Africa it is noted that almost all of the sampling is carried out using locally manufacturer “plastic type HD (Envirocon model GX1)” cyclone without any published knowledge of its size-selective performance as required by the original criteria (Grabe, 1988). The reason for the use of this particular cyclone head or the operating specifications such as flow rate of 1.9 or 2.2 Lpm could not be established. Interestingly, South Africa was the first country in the world to switch over to the new size-selective curve (Belle, 2004) and no amendments to the OEL to coal dust or silica dust has been made.

A French study (Gorner et al., 2001) of fifteen respirable aerosol samplers had studied the South African 25 mm cyclone had noted that the cyclones when operated at 1.9 lpm and 2.5 lpm flow rate, they conform to BMRC and ISO (1995) respirable curve with a D<sub>50</sub> of 5.81 microns and 4.21 microns respectively. Despite the above there appears to no regulatory guidance on operating these South African manufactured cyclones used for exposure monitoring.

A HSE size selection characteristic study (Kenny, Baldwin and Maynard, 1998) noted that the locally manufactured South African cyclones were very similar in performance to the HD type cyclone. The HSE tests were carried out at 1.8 Lpm, 2.0 Lpm and 2.3 Lpm with a resulting D50 of 5.91 microns, 5.01 microns and 4.61 microns respectively. There were no size-selective data for the SA cyclone that were readily available from the HSE (Kenny, 2016) to calculate the bias maps. However having without being tests carried out, HSE had recommended to operate the SA cyclone at flow rate of 2.2 Lpm and cyclones were being operated since 1997 (Belle and Du Plessis, 1998) to emulate ISO (1995) curve.
History of Australian gravimetric samplers

Since the adoption of gravimetric sampling in Australia in 1983, the plastic and aluminium HD cyclones have been used and operated at 1.9 Lpm. As per AS2985 (1987), Australian dust sampling followed the BMRC (Orenstein, 1960) with zero efficiency for particles of 7 microns. The AS2985 recommended dust sampling devices included the British Cast Iron Research Association (BCIRA), HD cyclones and SIMPED cyclones. However, AS2985 (2004, 2009) made amendments to the definition of the respirable dust aligned with the ISO (1995) definition and cyclones were recommended to be operated at 2.2 Lpm flow rate. Currently, further investigations have indicated that almost all of the sampling in some mining regions is carried out using a specific manufacturer, "plastic type HD" cyclone without any reference knowledge of its size-selective performance. The reason for the selection of this particular supplier of cyclones could not be established other than ease of its availability. In addition test evaluation reports about, conformity of the currently available SKC cyclone or Casella cyclone that are used in Australia are not readily available. Amendments to the OEL by switching over to the new size-selective curve have been made in NSW dust standards but not in QLD dust limits. The absence of publicly available original field evaluation data on switchover to the newly adopted curve has resulted in confusions over the validity of dust limits between the two states.

As a general observation, other than slight design modifications, currently available various cyclone particle elutriators are of the same design as that described by Higgins and Dewell (1966) used in the cyclone originally manufactured by the British Cast Iron Research Association. The South African, Australian and the UK SIMPEDS sampler are shown in Figure 2. The air inlet configuration of the SA cyclone sampler is different to the BGI, Casella and SKC cyclone samplers. It comprises a tangential slot entry rather than the tubular entry found on the other cyclones. The SA cyclone sampler is a sealed unit so the vortex finder is permanently attached to the cyclone elutriator.

![HD test samplers: SIMPEDS cyclone (Left); Australian cyclone (AS) (Middle) and South African cyclone (SA) (Right).](image)

In the absence of the original HSE (2016) data on South African cyclone tests (1998), it was decided to contact and obtain the raw test results carried out on SA samplers in 1996-97 from the French laboratory (Gorner, 2017) that recommended the sampler to be operated at 2.5 Lpm. Figure 3 shows the penetration efficiency of the cyclones for different flow rates using the French cyclone size characterisation tests. From the fractional penetration efficiency and Bias map at 2.5 Lpm for the HD type sampler (25 mm), it is noted that at 1.9 Lpm the cyclone largely oversamples both the BMRC and and ISO1995 respirable aerosol fraction. The French study recommended that the SA cyclone be operated at 2.5 Lpm to satisfy the requirement of D50=4 µm. The French study data noted that the SA cyclone didn't perform to Johannesburg curve at 1.9 Lpm nor the new ISO curve when operated at 2.2 Lpm. These conflicting French and UK studies necessitated the need for the review of penetration efficiency tests of South African cyclones for operations. In addition, the service providers or research laboratories both in Australia and South Africa could not demonstrate the exposure monitoring indeed meets the required ISO (1995) respirable dust sampling specifications.
CYCLONE SAMPLING EFFICIENCY AND DUST CONCENTRATION TEST

The cyclone sampling efficiency and dust concentration tests are very complex and require sophisticated laboratory test chambers, which are scarce with a shortage of expertise on operational monitoring experience. Currently, there are very few such facilities available globally such as in UK, France, Sweden and USA. Therefore, in the absence of such quality facilities in Australia and South Africa, tests were carried out independently at the HSE (UK). Tests were carried out to determine the penetration characteristics of a total of nine plastic cyclone samplers, 3 South African cyclones, three used Australian cyclones from three different mines and three new Australian samplers and three UK SIMPEDS sampler (HSE, 2016). The HSE tests are standard cyclone sampling efficiency tests with a well-defined protocol that can be repetitive and reproducible for evaluation purposes. For all comparison purposes, the UK SIMPEDS Casella plastic cyclone is considered as a 'true reference sampler' by the HSE. This is based on the previously well-established research study by Maynard and Kenny (1995) and the evaluation standard set forth by the HSE -UK (2000) to the mining industry. The design of the sampler test system is based on that described by Kenny and Liden (1991) used for the measurement of polydisperse aerosol penetration through cyclone samplers inside a calm air chamber and is not discussed here. The approach requires measurements of the aerodynamic size distribution of an aerosol penetrating through the cyclone sampler under test and that of the aerosol challenging it. The two size distributions are compared to obtain the penetration characteristics of the cyclone sampler. The experimental cyclone efficiency and dust concentration test set-up is shown in Figure 4. The laboratory confirmed that the cyclone flow rate before and after each test and was found to be within 2% of the target value of 2.2 Lpm.

DATA ANALYSIS AND CALCULATION OF SAMPLER BIAS FOR TEST AEROSOL

All the data associated with the evaluations in this study were obtained from the independently commissioned study at the HSE laboratories (UK). The reference SIMPEDS plastic cyclone and test cyclone sampler particle concentrations and particle size, and cyclone penetration was measured as a fraction of the reference aerosol. Using the
measured size data, fractional penetration efficiency were plotted to determine the D50 from the fitted curves for each of the test and reference cyclones. The measured performance data for each cyclone sampler was assessed against the respirable target convention defined in BS EN 481(1993). For the evaluation purposes, the bias values were calculated for the respirable aerosol size distribution range of 1 µm to 30 µm Mass Median Aerodynamic Diameter (MMAD) with up to 30 µm with geometric standard deviation (GSD) range of 1.75 to 4.0 (step of 0.25) as specified using the bias map approach in BS EN 13205-2 (2014). Using the calculated bias values, a two-dimensional diagram (bias map) showing the GSD and MMAD on the axes, and points of equal bias joined to form contours are drawn. In this paper the average of all the repeat tests for each of the gravimetric samplers were calculated and bias maps are produced for the identical calm test chamber conditions for test cyclones. For any aerosol size distribution A, the bias in the sampled concentration is defined in Workplace exposure – Assessment of sampler performance for measurement of airborne particle concentrations (BS EN 13205-2:2014) as:

\[
\Delta i = \Delta i (D_A, \sigma_A) = \frac{C_i - C_{std}}{C_{std}}
\]  

(1)

Where:

- Cstd is the concentration that would be sampled by a sampler that perfectly follows the sampling convention and is a function of the sampled aerosol size distribution, A;
- c is the correction factor stated either in the manufacturer’s instructions for use or in the relevant measuring procedure; No other correction factor may be applied to the sampled concentrations. If no correction factor is stated, c is assigned a value of 1.00.
- Ci is the mean sampled relative concentration and is a function of the sampled aerosol size distribution, A;
- DA is the mass median aerodynamic diameter of the sampled aerosol, A;
- \(\Delta i\) is the bias or relative error in the aerosol concentration measured using the candidate sampler, for aerosol size distribution A, and
- \(\sigma_A\) is the Geometric Standard Deviation (GSD) of the sampled aerosol, A.

Similarly, for the sampler flow rate of 2.2 Lpm, the Fractional Mass Sampled (FMS) from an aerosol with lognormal size distribution (aerodynamic mas median diameter, MMAD, and GSD, \(\sigma_g\)) will be a function of (Liden and Kenny, 1993) the size distribution parameters and the flow rate, Q, and is evaluated as an integral over all aerodynamic particle sizes, Dae,

\[
FMS(MMAD, \sigma_g, Q) = \text{corr} \int_0^\infty \text{eff} (D_{ae}, Q) f(D_{ae}, MMAD, \sigma_g) dD_{ae}
\]  

(2)

Where:

- \(\text{eff} (D_{ae}, Q)\) = the sampler efficiency curve, including measured or assumed aspiration losses.
- \(f(D_{ae}, MMAD, \sigma_g)\) = the mass distribution density function of an aerosol with parameters MMAD and \(\sigma_g\)
- \(\text{corr}\) = a correction factor used to overcome sampler bias.

The sampler bias is then calculated (Liden and Kenny, 1993) for each aerosol size distribution selected, and each flow rate (in this study at 2.2 Lpm), by comparing the numerically modelled FMS values to what would have been obtained by an ideal sampler perfectly following a sampling convention,

\[
\text{bias}(MMAD, \sigma_g, Q) = 100 \frac{FMS_{\text{SAMPLER}} - FMS_{\text{IDEAL}}}{FMS_{\text{IDEAL}}}
\]  

(3)
RESULTS AND DISCUSSION

Table 1 shows the measured particle size at which 50% of the particles penetrated for all test cyclones ($D_{50}$) for each test, along with the percentage deviation, Bias $D_{50}$, from the $D_{50}$ given in EN 481/ISO 1995 (4 μm). Figure 5 shows the average fractional penetration curve for the three South African, six Australian (3 used and 3 new) and three SIMPEDS gravimetric samplers operated at 2.2 Lpm flow rate. The plot also highlights the ISO (1995) respirable convention, defined in EN 481 (1993), for comparison.

Table 1: Summary statistics of measured $D_{50}$ of SIMPEDS, Australian and South African samplers.

<table>
<thead>
<tr>
<th>Gravimetric Sampler</th>
<th>Test-1</th>
<th>Test-2</th>
<th>Test-3</th>
<th>Average</th>
<th>SD</th>
<th>RSD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$D_{50}$, μm</td>
<td>% Bias $D_{50}$</td>
<td>$D_{50}$, μm</td>
<td>% Bias $D_{50}$</td>
<td>$D_{50}$, μm</td>
<td>% Bias $D_{50}$</td>
</tr>
<tr>
<td>SA1</td>
<td>5.51</td>
<td>37.75</td>
<td>5.36</td>
<td>34.00</td>
<td>5.33</td>
<td>33.25</td>
</tr>
<tr>
<td>SA2</td>
<td>5.87</td>
<td>46.75</td>
<td>5.68</td>
<td>42.00</td>
<td>5.82</td>
<td>45.50</td>
</tr>
<tr>
<td>SA3</td>
<td>4.62</td>
<td>15.50</td>
<td>4.68</td>
<td>17.00</td>
<td>4.61</td>
<td>15.25</td>
</tr>
<tr>
<td>AS1*</td>
<td>5.80</td>
<td>45.00</td>
<td>5.48</td>
<td>37.00</td>
<td>5.64</td>
<td>41.00</td>
</tr>
<tr>
<td>AS2*</td>
<td>6.13</td>
<td>53.25</td>
<td>6.36</td>
<td>59.00</td>
<td>6.10</td>
<td>52.50</td>
</tr>
<tr>
<td>AS3*</td>
<td>6.20</td>
<td>55.00</td>
<td>5.85</td>
<td>46.25</td>
<td>6.00</td>
<td>50.00</td>
</tr>
<tr>
<td>AS1**</td>
<td>5.70</td>
<td>42.50</td>
<td>5.42</td>
<td>35.50</td>
<td>5.42</td>
<td>35.50</td>
</tr>
<tr>
<td>AS2**</td>
<td>6.28</td>
<td>57.00</td>
<td>6.14</td>
<td>53.50</td>
<td>6.04</td>
<td>51.00</td>
</tr>
<tr>
<td>AS3**</td>
<td>5.67</td>
<td>41.75</td>
<td>5.65</td>
<td>41.25</td>
<td>5.71</td>
<td>42.75</td>
</tr>
<tr>
<td>SIMPEDS S1</td>
<td>4.40</td>
<td>10.00</td>
<td>4.36</td>
<td>9.00</td>
<td>4.31</td>
<td>7.75</td>
</tr>
<tr>
<td>SIMPEDS S2</td>
<td>4.39</td>
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<td>4.33</td>
<td>8.25</td>
<td>4.28</td>
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<tr>
<td>SIMPEDS S3</td>
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<td>4.50</td>
<td>4.18</td>
<td>4.50</td>
<td>4.12</td>
<td>3.00</td>
</tr>
</tbody>
</table>

* Used Australian samplers from different mine sites; ** New Australian sampler

From Table 1, it is noted that the $D_{50}$ value for South African, Australian and the SIMPEDS samplers were 5.28, 5.95 and 4.28 microns respectively. From the tabulated results, it can be clearly seen that cyclone samplers SA1 and SA2 exhibited a higher positive sampling bias (>35%) compared to the respirable convention (BS EN 481, 1993) for theoretical aerosols with mass median aerodynamic diameters 1–30 μm and geometric standard deviations 1.75-4. They would therefore be expected to overestimate measurements of the respirable dust in the field. However, SA3 sampler exhibited unusually lower bias of <20%, but demonstrated significant variations between the other two in terms of sampling performance.

The reason for the higher $D_{50}$ cut-point and high bias for SA1 and SA2 is not clear as the cyclone specifications are not readily available. Laboratory inspections concluded that there was an observed difference in the air inlet slot dimension for SA2, which appeared to be wider than SA1 and SA3 whose dimensions appeared similar. It was noticed that the SA1 sampler had been extensively used as there were signs of wear to the body. Whether these observations were the cause of the differences in sampling performance is speculative, but suggest a potential variation in manufacturing tolerance. On the other hand, the measured D50 of the Australian gravimetric sampler (both new and used) was up to 59% higher (AS2 in Test2) than the target value of 4 μm.
From the Table 1 and Figure 5 it is noted that the measured $D_{50}$ for the South African samplers was considerably higher than the target value of 4 µm (given in BE EN 481) i.e. between 33% and 47% higher, except for SA3 sampler with a $D_{50}$ of 4.64 microns for 3 repeat tests. Similarly, measured $D_{50}$ for the Australian (used and new) cyclones was considerably higher than the target value of 4 µm (given in EN 481), i.e. between 35% and 59% higher. In contrast, the measured $D_{50}$ for the ‘true reference’ UK SIMPDES plastic cyclone was much closer to the target value i.e. 3 – 10 % higher with an average $D_{50}$ of 4.28 microns. It is also interesting to note that there are differences in individual Australian and South African cyclone samplers or larger scatter in terms of measured $D_{50}$ values given by a higher coefficient of variation (RSD) in Table 1. Similarly, what is a critical finding from the penetration plots (Figure 5) is that both the South African and Australian sampler is that the tail of the penetration graphs also extends much further than the reference SIMPDES UK ‘true’ cyclone samplers i.e. the penetration approaches zero at about 8 µm for the SIMPDES and > 15 µm for the South African and Australian cyclones respectively.

Figure 6 shows the bias maps of gravimetric samplers (South Africa-Top); Australian Sampler (Middle) and SIMPDES sampler (Bottom). Both the South African and Australian gravimetric samplers exhibited a high sampling bias, giving a positive bias often greater than 30% higher than the respirable convention (EN 481, 1993). They would therefore be expected to overestimate measurements of the respirable concentration of airborne dust in the workplace.

**Figure 5:** Fractional penetration average of particles through South African (left), Australian (middle) and UK SIMPDES cyclone (Right) gravimetric samplers as a function of aerodynamic particle diameter (HSE, 2016).

**Figure 6:** Bias map of gravimetric samplers (South Africa-Top); Australian Sampler (Middle) and SIMPDES sampler (Bottom) for various dust.
These independent laboratory results with higher $D_{50}$ values and bias have reinforced the conclusions that the current South African and Australian gravimetric samplers significantly overestimated the measured respirable dust levels based on size analyses during field measurements (Belle, 2017). Regardless of the attributable reasons for the non-conformance to the ISO (1995) size-selective curve, both the current South African and Australian cyclones must be discontinued from use in their current design.

**MEASURED CYCLONE DUST CONCENTRATIONS**

Table 2 and Figure 7 show the dust levels measured by each gravimetric test sampler when exposed to an airborne coal dust cloud in the HSE laboratory test chamber. It can be seen that the samples of each SIMPEDS and Australian cyclone sampler gave consistent measurements of the dust concentration, given by a coefficient of variation (RSD) value of less than 5% between samplers for each repeat test. However, the SA cyclone showed a significant variation in performance between samplers illustrated by a RSD of 13.3%, 19.2% and 21.5% for each test. This supports the variation in $D_{50}$ between the three SA cyclone samplers shown in Table 1. The ratio of SA cyclone sampler dust levels to average SIMPEDS cyclone sampler dust levels increased in the order SA3 (0.99), SA1 (1.13), SA2 (1.40). This is consistent with the increase in $D_{50}$ values shown in Table 1 with only SA3 sampler closely matching the SIMPEDS sampler. The ratio of Australian cyclone dust measurement to SIMPEDS cyclone dust measurement is consistently around 1.41 - 1.53, i.e. the Australian cyclone sampler measured approximately 40 - 50% higher dust levels than the reference SIMPEDS cyclone sampler. This is consistent with the higher value of $D_{50}$ measured previously for all three used and new Australian cyclones.

From the results it was noted that the measured dust levels of gravimetric samplers are significantly different when operated at the same sampler flow rates. The average measured dust levels for the SIMPEDS, South African and Australian samplers for the sampling period were 6.71 mg/m$^3$, 9.79 mg/m$^3$ and 7.87 mg/m$^3$ respectively. Using the linear regression of the data, it can be inferred that there is a positive ‘concentration measurement bias’ in respirable dust levels for Australian and South African samplers by 46% and 26% respectively at the current coal dust compliance limit. The implication of this finding is significant where there exist an open-ended compliance determination process without specific and transparent guidance mechanisms for review.

**Table 2: Summary of measured dust levels under controlled coal dust tests**

<table>
<thead>
<tr>
<th>Test#</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mg/m$^3$</td>
<td>Avg</td>
<td>SD</td>
</tr>
<tr>
<td>SA1*</td>
<td>8.4</td>
<td>7.2</td>
<td>7.0</td>
</tr>
<tr>
<td>SA2*</td>
<td>10.2</td>
<td>8.9</td>
<td>1.1</td>
</tr>
<tr>
<td>SA3*</td>
<td>8.03</td>
<td>6.2</td>
<td>5.7</td>
</tr>
<tr>
<td>AS1**</td>
<td>10.7</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>AS2**</td>
<td>11.5</td>
<td>10.9</td>
<td>0.5</td>
</tr>
<tr>
<td>AS3**</td>
<td>10.6</td>
<td>8.9</td>
<td>8.9</td>
</tr>
<tr>
<td>RC1***</td>
<td>7.4</td>
<td>6.4</td>
<td>6.4</td>
</tr>
<tr>
<td>RC2***</td>
<td>7.7</td>
<td>7.5</td>
<td>0.2</td>
</tr>
<tr>
<td>RC3***</td>
<td>7.3</td>
<td>6.5</td>
<td>6.1</td>
</tr>
</tbody>
</table>

*South African HD sampler; ** Australian HD sampler.
CONCLUSIONS

This paper summarises comparative cyclone penetration efficiency and dust concentration results evaluated under controlled conditions between the South African, Australian and the ‘reference true’ SIMPEDS UK reference sampler operated in accordance with the ISO (1995) size-selective curve at a flow rate of 2.2 Lpm. The following conclusions can be drawn from the sampler evaluations:

- An independent particle penetration efficiency results showed that the measured D50 for the ‘true reference’ UK SIMPEDS standard plastic cyclone was much closer to the target value i.e. 3 – 10 % higher with an average D50 of 4.28 microns.
- In contrast, particle penetration efficiency results showed that the D50 value for South African and the Australian samplers were 5.28, and 5.95 microns respectively. It can be clearly shown that the South African samplers exhibited a higher positive sampling bias, than the target value of 4 µm (given in BE EN 481, 1993) i.e. between 33% and 47% higher, except for SA3 sampler with a D50 of 4.64 microns for 3 tests. Similarly, measured D50 for the Australian (used and new) cyclone samplers was considerably higher than the target value of 4 µm, i.e. between 35% and 59% higher.
- Based on the particle size penetration plots, it is noted that in both the South African and Australian samplers is that the tail of the penetration graphs also extends much further than the reference SIMPEDS ‘true’ cyclone samplers i.e. the penetration approaches zero at about 8 µm for the SIMPEDS and > 15 µm for the South African and Australian cyclones respectively.
- Calculated average bias maps were prepared using the sampling efficiency data, it is noted that both the South African and Australian gravimetric samplers exhibited a high sampling bias, giving a positive bias often greater than 30% higher than the respirable convention. They would therefore be expected to overestimate measurements of the respirable dust levels in the workplace.
- An independent concentration measurement of dust aerosol showed that the average measured dust levels for the SIMPEDS, South African and Australian samplers for the sampling period were 6.71 mg/m³, 9.79 mg/m³ and 7.87 mg/m³ respectively. Using the linear regression of the data, it can be inferred that there is a ‘concentration measurement bias’ in respirable dust levels for Australian and South African samplers by 46% and 26 % respectively at the current coal dust compliance limit. The implications of this finding are significant where there exist an open-ended compliance determination process without specific and transparent guidance mechanisms for review.

In summary, independent laboratory data and their analyses with higher D50 values and bias have reinforced the conclusions that the current South African and Australian gravimetric...
sampler results significantly overestimated the measured respirable dust levels. It is acknowledged that the manufacturing challenges of sampler design variations, inlet geometry variations of samplers, sampler material type, and some discrepancies in evaluation methodology difficulties in particle-size dependent efficiency measurement are well understood by the aerosol professionals. However, they should not be the reason in over or underestimation of the personal exposure results and also cause significant non-compliance and loss of confidence in the exposure data that ultimately gets used in deriving dose-response relationships. This situation can be avoided by following the path of single SIMPEDS sampler in the South African and Australian industry as practiced in USA. The benefits of harmonised use of a single true standard sampler would enable greater understanding of exposure data derived worldwide or within the mining industry. Regardless of the attributable reasons for the non-conformance to the ISO (1995) size-selective curve, both the current South African and Australian cyclones must be discontinued for use in their current design. The study has demonstrated that it is critical to ensure the samplers used at the operations by the third party service providers or research laboratories for exposure monitoring indeed demonstrate and meet the ISO (1995) performance criteria and quality.

ACKNOWLEDGEMENTS

The author would like to thank all the reviewers for their constructive comments and encouraging remarks. Also, appreciates Mr. A. Thorpe, L Kenny of HSE who carried out the experimental work and the opportunities to witness the cyclone evaluations and technical exchanges on laboratory work. Hopefully the findings in this paper will assist in ensuring the selection and use of the samplers and more importantly, result in the implementation of the harmonised use of prescribed dust sampler for use in the global mining industry. This paper and the work contained herein is an effort to improve the exposure monitoring and improve engineering controls in workplaces.

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RESEARCH, DEVELOPMENT AND APPLICATION OF DUST SUPPRESSION TECHNOLOGY

Jon Roberts¹ and Peter Wypych²

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 high-energy micro-mist sprays. The utilisation of CFD modelling and simulation 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 flow of dust emissions and for the development of new systems that can be used in the coal industry. CFD modelling is shown to be effective for modelling both spray dispersion and cross-wind effects allowing designers to develop high-efficiency dust suppression systems with a greatly improved level of confidence when compared to traditional techniques. New high-energy micro-mist technology is also outlined as a key element in developing high-efficiency dust suppression systems. Experimental results and industry applications are presented to demonstrate the significant improvements that can be achieved in suppressing or reducing airborne dust emissions.

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. One of the primary control methods implemented today consists of water sprays designed to wet material as a way of limiting dust release, 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 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 two specific operational areas that have been identified as troublesome for the control of dust (among many others); run-of-mine (ROM) dump hoppers and stockpile stackers. These operations can generate relatively large quantities of dust in conditions, which can be described as challenging at best. The outdoor nature of these operations means that they are particularly susceptible to wind disturbances and the high drop heights that are present result in the development of fast moving and “dense” dust clouds. 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

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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).

OPTIMISING DUST CAPTURE EFFICIENCY

To optimise the efficiency of a dust control/suppression system the key influencing variables must first be identified. Generally, the first component selected in the design of a dust suppression system is the nozzle type which is selected based on Droplet Size Distribution (DSD), spray penetration, mist density, spray angle, and droplet velocity. These are all a function of spray nozzle design and input conditions (e.g. water pressure). The aspect of a nozzle’s performance most important to its effectiveness at capturing dust is droplet size. The reason for this is related to Stokes’ law, which describes the frictional force or drag forces that are exerted on a spherical object moving in a viscous fluid. As both a droplet and a dust particle travel through the air they will impart a force on the air which will alter their motion and effect their interactions with each other. Figure 1 depicts this interaction. If a very small object is travelling towards a larger object, the fluid flowing around the large object will impart a force on the small object causing it to become entrained in the disturbed air and travel around the object rather than impacting with it. The greater the difference in size the more pronounced this effect will be and as such the less likely it is that impact will occur. Based on this explanation it can be concluded that to maximise the potential of dust capture by water droplet it is most important to have droplets of similar size to the dust particles being captured. Furthermore, as well as having droplets of similar size, it is also necessary to have droplet concentration or mist density greater than the concentrations of dust in the air such that all the dust can be captured.

![Diagram of Particle-droplet interaction](image)

Given this knowledge, it is now possible to analyse the performance of systems currently in use. There are two forms of spraying systems that are commonly used for dust suppression: systems using water only sprays generally at relatively low pressures producing a coarse droplet spray, or systems using air atomising nozzles producing a very fine low-velocity mist. These systems have their own advantages and limitations dependent on the conditions. Most water only spraying systems produce a relatively low concentration of coarse droplets with an aerodynamic diameter in the range of 200-600 microns (Yao-she, Gao-xian, Jun-wei, & Xiao-bo, 2007). The coarse droplets result in a spray that can handle crosswinds without too much deviation and can span large distances, however, the large droplet size and low concentration do not result in good capture efficiencies as fine dust particles easily escape through the space between the droplets resulting in minimal dust-to-droplet impact. On the other hand, air atomised sprays produce very fine droplets of a relatively high concentration, which are extremely effective at capturing fine dust particles ($d_{50} < 50$ microns). These air atomisers, however, do not deal well with significant air flow or highly concentrated dust clouds and are easily blown away in these conditions resulting in poor dust capture performance. Furthermore, air atomisers also add air into the dust-air problem that is already present. The
issues presented here plague the performance of many systems typically found in coal mines and go some way to explaining the poor performance of many of the systems currently installed in the industry.

To develop more efficient systems a solution must be found that consists of sprays with correctly sized droplets, high droplet concentrations, and with sufficient energy to withstand adverse air flow conditions. These requirements led to the development of high energy micro mist nozzles (EnviroMist). These nozzles operate at pressures ranging typically from 100-300 bar and produce a highly dense and finely atomised spray with high capture effectiveness even in adverse conditions such as high cross winds. The comparison below (Figure 2) shows a standard water only spray versus a high energy micro mist both captured in one ten-thousandth of a second. The first thing to note is the droplet size, the traditional spray produces coarse droplets which are easily visible in the image and the droplet size distribution shows the droplet diameter (DV50) of micro-mist spray being less than one sixth of the coarse droplet spray. These coarse droplets are much larger than the dust particles they aim to capture and as a result they are generally very inefficient. Secondly, the two sprays are vastly different in their droplet concentration although having similar water consumption. This is a function of the decreased droplet size resulting in millions more droplets dramatically increasing the probability of capture. The two sprays shown in Figure 2 have the same water consumption, however, given the improved capture efficiency of these sprays, it has been found that they can be operated at flow rates as much as 50% lower than current systems. This is significant given the importance of sustainable water usage rates; it has been reported that up to 70% of the water supply in the Pilbara is used for mining activities with 50% of this being used for dust suppression purposes (Mills, 2010).

![Figure 2: Comparison of coarse-droplet and fine droplet micro-mist sprays](image)

**DESIGN METHODOLOGY**

The effectiveness of airborne dust suppression systems can be significantly improved by understanding the physics involved as described above and by making basic considerations based on readily available data. Considerations that need to be made are summarised below:

- Environmental factors
- Size of focus area
- Wind velocities
- Water consumption/availability
- Physical barriers
- Hazards (such as machine or equipment movements and flying rock,
- Dust properties, including Particle Size Distribution (PSD)
- Velocity
• Concentration
• Nozzles Selection and Position
• Positioned for maximum coverage, maintenance access, and free of any interference issues that could cause damage
• Nozzle selection for maximum capture efficiency

Nozzle selection and positioning are dependent on both the dust properties and the environmental factors. Nozzles need to be selected so that they produce water droplets of similar size to the dust particles to ensure effective capture (as described earlier) — this is achieved by proper selection of both the nozzle and its operating conditions (higher pressure leads to smaller droplets but increased water consumption for a specific nozzle). The velocity of the mist is also of vital importance and is dictated by the choice of nozzle and the operating parameters; this should be selected based on the area to be covered and the energy of air and dust flows that will be encountered by the mist. The position should be determined based on maximum coverage of the dust-air flow region whilst ensuring the nozzles are safe from damage — this can be easily determined using a 3D-CAD package in combination with mist profile data.

Matching droplet size with dust particle size has been demonstrated in Figure 3 (Wypych, Hastie, Wangchai, & Grima, 2015), allowing optimal capture efficiency of the dust particles by the water droplets.

![Figure 3: PSD of dust compared with water DSD](Wypych, Hastie, Wangchai, & Grima, 2015)

Measurement of air and dust flows that occur in the application is also of vital importance, making sure to consider the worst-case scenario and the common case to find an acceptable compromise. This data can be considered as a design variable for selection of an appropriate nozzle and/or the correct operating conditions for a selected nozzle. The effect that dust-air flow in an area has on the mist produced by a specific nozzle can be investigated by using numerical modelling methods, specifically CFD simulations. An investigation and description into the use of CFD for this purpose is presented in the next section. The penetration distance of the mist under varying conditions is the key parameter that should be measured using this technique to ensure that the mist maintains the desired coverage under the required conditions.

Once a nozzle and its operating conditions are selected, the mist should be modelled using 3D-CAD software with a basic model of the site. This allows the nozzle positioning to be determined and more easily visualised ensuring full coverage of the desired area whilst maintaining maintenance access and protection from damage. An example of this is shown in Figure 4, a full coverage high energy spraying system designed for a standard Run-of-Mine (ROM) bin. The system is designed to minimise possible damage from falling rocks or machinery in the area, as well as providing full mist coverage appropriate to the application.
Figure 4: Dust suppression system for ROM bin

NUMERICAL MODELLING

Numerical modelling of the sprays used for dust suppression purposes presents the opportunity to understand the flow dynamics and develop a solution with confidence versus the trial and error based approaches that are commonly used. Research has been completed to develop a validated and best practice approach of using CFD to model sprays under varying conditions. ANSYS Fluent is the primary CFD package that has been used; Fluent provides Volume-of-Fluid (VOF), Eulerian and mixture models as well as a discrete phase model that uses Lagrangian trajectory tracking. There are two methods that are commonly used for spray modelling, Eulerian-Eulerian or Lagrangian-Eulerian. The volume-of-fluid method uses the Eulerian reference frame and is a free surface modelling technique which allows fluid-to-fluid interfaces to be modelled this is important for the modelling of the interaction of droplets with air. The VOF method requires an extremely fine mesh to model the breakup and motion of very small droplets and as such is extremely computationally expensive. 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.

The coupled Lagrangian trajectory tracking method was chosen for this application primarily due to the reduced computational expense whilst maintaining acceptable accuracy. This model injects discrete particles into the continuous flow field and tracks the particle trajectory by integrating a force balance on each particle per the Lagrangian reference frame. The force balance can be written as:

\[
\frac{du_p}{dt} = F_D(u - u_p) + \frac{g_x(\rho_p - \rho)}{\rho_p} + F_x
\]

This equates the particle inertia with the forces acting on the particle. Where \(F_x\) is the acceleration term, \(F_D\) is the drag force, \(u\) and \(u_p\) are the fluid phase and particle velocities respectively and, \(\rho\) and \(\rho_p\) are the fluid and particle densities.

Critical to the accuracy of this model is the meshing method and size used. Two mesh types were investigated; a polyhedral cell mesh and a Cartesian cut cell mesh both with refinement close to the nozzle exit. Previous literature modelling of sprinkler sprays found that a grid resolution of 75 mm was the largest that could be used (Husted, 2007) whilst maintaining accuracy. A mesh independence study was conducted for this application and found that the grid resolution should be less than 30 mm in the far field and less than 5 mm near to the nozzle, with minor difference in accuracy between cut cell and polyhedral meshing methods found. The use of the polyhedral cells did, however, produce a 30-40% reduction in mesh size. For this application, the two-equation realisable k-epsilon model (Shih, Liou, Shabbir, Yang, & Zhu, 1995) was chosen due to its improvement over the commonly used standard k-epsilon model and its optimisation for modelling free-stream turbulence which is most relevant.
Two-way modelling is used to accurately account for the interaction between the air and droplets, the Stokes-Cunningham drag model is used as it provided the most accurate result for the spray being modelled.

To establish a validated model extensive experimental research on spray dynamics has been undertaken. This includes measurement of velocity, profile and penetration of many of the micro mist nozzles available under varying conditions. This allows for validation of the numerical model primarily in terms of the turbulence and drag models applied such that the flow dynamics correlates well with real world data.

**Model Validation**

To ensure the accuracy of simulations it is important that the models are validated against experimental data. There are two conditions that are used to validate the models; a single spray operating in static conditions and a single spray operating under a cross-wind. Experimental data is collected using a variety of laser and imaging techniques with droplet/mist velocity and position the key variables being measured. The data which is shown in Figure 5 was collected using Laser Doppler Velocimetry (LDV); this is the droplet velocity along the centreline of the stream in static conditions. The discrete phase mean cell velocity with distance predicted by the simulation model is also shown in Figure 5. It can be seen that the velocity is slightly over predicted by the simulation however it provides a reasonable estimate allowing the prediction of mist velocity and penetration that can significantly improve the nozzle positioning process.

![Figure 5: Measured Mean Centreline Velocity of Enviromist Nozzle 1](image)

Figure 6 shows the spray profile produced by the simulation compared with the expected result observed experimentally. Due to the turbulent nature of an atomised spray this profile can be variable, however, acceptable agreement is found between the predicted and experimental data.
The second condition that is considered to ensure correlation between experimental and predicted data is the spray geometry under cross-wind conditions. This is achieved through image analysis allowing us to measure the deflected profile of the spray due to the cross-wind. Figure 7 shows the experimental data presented as a measurement of the distance of the spray front from the nozzle exit in the horizontal and vertical directions. Figure 8 shows the predicted mist deflection/predicted profile of one nozzle as simulated using Fluent. There is still some analysis and testing required to get this model to the accuracy required without too much computational expense however the ability to predict this deflection will go a great way to ensuring that nozzles can be correctly selected for conditions associated with any specific application.

**Figure 7: Measured Mist Deflection**

![Mist Deflection in Crosswind](image)

<table>
<thead>
<tr>
<th>Distance from Nozzle Parallel to Spray (mm)</th>
<th>Distance from Nozzle Perpendicular to Spray (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>500</td>
<td>500</td>
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<td>4000</td>
<td>4000</td>
</tr>
</tbody>
</table>

**Figure 6: Spray Profile - Experiment vs Prediction (Enviromist Nozzle 2)**

![Spray Profile](image)
APPLICATIONS

It was identified at the outset of this paper that ROM bins and stockpile stackers are two problematic areas 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 should 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.

A typical ROM bin using a non-optimised dust suppression system with low-pressure coarse droplet water sprays can consume water at a rate of 1000 L/min with dust capture efficiencies of less than 30% (Courtney and Cheng, 1977). A system utilising micro-mist nozzles, designed with CAD and CFD modelling techniques was installed at a mining operation in late 2015 (modeled in Figure 4). This mine was suffering from significant dust issues with their existing system delivering little to no effective dust capture on the ROM bin. The installed system delivered 100% airborne dust capture and suppression at a water consumption of only 300 L/min, a 700 L/min reduction compared to the original system that was installed.

Other recent applications of this dust suppression technology include: mobile crushing station and stacker (Figure 9); BSL boot end discharge in underground coalmine (Figures 10); grab bucket ship unloading (Figure 12).
Figure 10: BSL boot end discharge (with new dust suppression)

Figure 11: Effectiveness of new dust suppression for BSL

Figure 12: Grab bucket ship unloading (before and after dust suppression)

Stockpile stackers are an area where this technology has not yet been applied. Typically, the control of dust in this scenario is very difficult and generally limited to material wetting on the stacker and surface treatments of the stockpile itself to avoid dust lift-off. The large drop height combined with high winds results in vast amounts of dust being generated. CFD can be used to predict both the air/dust flow as well as the performance of sprays selected for use in this area. Optimisation in this manner can lead to a greater improvement in the initial performance of the system limiting the amount of modification required after installation.
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 CFD modelling have been identified as enabling technologies for improving the control and capture of airborne dust. The fundamental principles contributing to effective dust capture by water spraying type systems have to be identified and analysed. 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. For this reason, a new technology has been presented in the form of high-energy micro-mist nozzles developed by EnviroMist. These nozzles provide high-velocity and high-density fine droplet sprays well suited to the conditions found in mining operations. The use of CFD technology is presented as a means of improving the design and performance prediction of airborne dust suppression systems. A validated model is presented enabling the simulation of sprays in typical conditions allowing prediction the spray dynamics under the expected conditions specific to each individual application. It is expected that this will allow the time taken to design an airborne dust suppression system to be significantly reduced and much higher dust capture efficiencies to be achieved once installed. Finally, some potential future applications are suggested.

ACKNOWLEDGEMENTS

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REFERENCES

IMPLEMENTATION OF INTERACTIVE SPONTANEOUS COMBUSTION HAZARD ASSESSMENT AND MANAGEMENT AT MEANDU MINE

Basil Beamish¹, Dave Edwards², Jan Theiler³

ABSTRACT: Since recording a spontaneous combustion event in-pit at Meandu Mine in 2014 interactive spontaneous combustion hazard assessment and management has been adopted by the mine to eliminate the risk of any future events. This has proved to be very successful due to a diligent and systematic set of management practices that commences with sampling and spontaneous combustion testing of the coal as it is exposed in each new strip at the mine, with feedback of results into the mine planning process through to stockpiling of the coal at the mine and at the Tarong Power Station. The key control in this process is the use of a new test method that quantifies the minimum incubation period for spontaneous combustion to develop under site conditions.

INTRODUCTION

Leading practice in all coal mines requires the implementation of a Principal Mining Hazard Management Plan (PMHMP) for spontaneous combustion. In Australia, sound scientific data is a mandatory requirement for use as a basis to assess the spontaneous combustion hazard. This information is routinely used for developing risk assessments, control measures, standard operating procedures and management strategies in order to provide safer working environments for personnel, reduced potential for lost time and decrease asset losses.

Australian coalmine operations have been using adiabatic oven testing since the early 1980’s to rate the propensity of coal to spontaneously combust (Humphrey, Rowlands and Cudmore 1981). The R⁷₀ initial self-heating rate parameter normally obtained from these tests is rated using a relative scale (Beamish and Beamish 2011). Evaluation of the resulting intrinsic spontaneous combustion propensity rating enables a general appraisal of the possible likelihood of a spontaneous combustion event to be made; however it provides no indication of the timeframe in which an event can occur under the prevailing site conditions. A more advanced method of adiabatic testing is now available that replicates initial site conditions and measures the incubation behaviour of the coal, thus overcoming the major deficiencies of current spontaneous combustion propensity index parameters (Beamish and Theiler, 2017). The results obtained are benchmarked (calibrated) against actual on-site spontaneous combustion events of a range of coals from Australia and overseas covering the full rank spectrum from low to high rank.

Since recording a spontaneous combustion event in-pit in May 2014 (Beamish, Edwards and Theiler, 2015), Meandu Mine has routinely implemented an interactive spontaneous combustion hazard assessment strategy that utilises adiabatic incubation testing to help guide the management of safe coal extraction, handling and stockpiling from the mine face to the Tarong Power station. This paper presents operations for future reference examples of the results obtained from using this strategy, which could be used by other.

MINE LOCATION AND GEOLOGICAL SETTING

Meandu Coal Mine is located in the late Triassic Age (~200-220 Ma) Tarong Basin approximately 200 km north west of Brisbane, Queensland (Figure 1). A thermal product coal is produced from three main seams in the Tarong Beds to supply Stanwell Corporation’s Tarong Power Station (TPS) (Simmons, Edwards and Ferdinands, 2013). The Tarong Beds coal measures consist of thick coal seams which are interlayered with fine grained clay.
quartz-rich or iron-rich stone bands of variable thickness within a sedimentary package of conglomerates, sandstones, siltstones and mudstone. Pegrem (1995) previously classified the coal from the Tarong Beds as high ash, low volatile, low sulphur steaming coal with weak coking properties. There are three main seams being mined at Meandu Mine, namely Ace, King and Queen. The seams increase in ash content with stratigraphic depth (Young, 2013). As the coal at Meandu Mine is not able to be easily “free dug” it is blasted in order for it to be mined.

There are three main seams being mined at Meandu Mine, namely Ace, King and Queen. The seams increase in ash content with stratigraphic depth (Young, 2013). As the coal at Meandu Mine is not able to be easily “free dug” it is blasted in order for it to be mined.

Figure 1: Tarong Basin Location (Queensland Department of Mines and Energy, 2013)

COAL SAMPLES AND ADIABATIC TESTING

Coal samples were collected from various strips at Meandu Mine as they were uncovered (Figure 2), as well as two samples (C06 King B and Carey’s 2 King) from stockpiled coal. The samples were tested in an adiabatic oven to establish their $R_{70}$ values and minimum incubation period. The $R_{70}$ testing procedure is described by Beamish (2005) and essentially involves testing a dried, crushed coal sample under adiabatic conditions from a fixed starting temperature of 40°C. The Incubation test procedure uses the coal in its “as-mined” moisture state and the test commences from a starting temperature that reflects the site-specific conditions. In the case of Meandu Mine a typical summer temperature of 35°C is used. The results obtained provide both an indication of the minimum incubation period (the time taken to reach thermal runaway) and the characteristic behaviour of the coal as self-heating progresses. The results are benchmarked against the heating that occurred in a coal bench (K4E ST12) at Meandu Mine in May 2014.
Figure 2: Sampled coal material from Ramp 5 ST7 for spontaneous combustion testing

The coal quality details of the samples are contained in Table 1 and their similarity in coal rank is demonstrated on a Suggate rank plot (Suggate, 2000) shown in Figure 3. According to the ASTM coal rank classification the coal is high volatile B bituminous. The coal type is predominantly high in inertinite (dull) as most of the samples plot below the New Zealand high vitrinite coal band (Figure 3).

Table 1: Coal quality data and ASTM rank rating for coal samples from Meandu Mine

<table>
<thead>
<tr>
<th>Sample</th>
<th>Moisture (%, ar)</th>
<th>Ash (% db)</th>
<th>Volatile Matter (% dmmf)</th>
<th>Calorific value (Btu/lb, mmmf)</th>
<th>ASTM rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>K4E ST12 AK</td>
<td>7.3</td>
<td>8.7</td>
<td>40.2</td>
<td>13666</td>
<td>hvBb</td>
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<tr>
<td>C06 King B</td>
<td>8.0</td>
<td>15.6</td>
<td>35.7</td>
<td>13906</td>
<td>hvBb</td>
</tr>
<tr>
<td>Carey’s 2 King</td>
<td>6.5</td>
<td>17.5</td>
<td>34.8</td>
<td>13837</td>
<td>hvBb</td>
</tr>
<tr>
<td>K4S ST3 Q</td>
<td>7.4</td>
<td>10.4</td>
<td>36.9</td>
<td>13868</td>
<td>hvBb</td>
</tr>
<tr>
<td>Ramp 5 ST4 Q1</td>
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</table>

Figure 3: Suggate rank plot of coal samples from Meandu Mine

ADIAATIC SELF-HEATING RESULTS AND DISCUSSION
R70 values

The R70 values for the Meandu coal samples are shown in Figure 4. Generally, as the ash content of the coal increases the R70 value decreases. However, the Carey’s 2 King sample was from a stockpile that had been formed for some time before being sampled. Consequently, it had undergone some pre-oxidation, which is indicated by the lower R70 value for the ash content of the coal. The fresh, low to medium ash content coal would be classified as having a high intrinsic spontaneous combustion propensity, whereas the high ash content and pre-oxidised coal has a medium intrinsic spontaneous combustion propensity rating.

Incubation behaviour

Since the R70 test is performed on a dry-basis it does not provide any indication of the moderating influence of moisture in the coal on the low temperature self-heating behaviour. However, the Incubation test results in Figure 5 clearly show the moisture moderating effect. For example, the Meandu samples C06 King B, Ramp 5 ST4 Q1 and Ramp 5 ST4 Q2 all have similar R70 values (3.14, 3.37 and 3.39 respectively rating them as having a high intrinsic spontaneous combustion propensity), but it is only the Ramp 5 ST4 Q1 sample that has a short incubation period to thermal runaway. This sample has a moisture content of 5.4%, whereas the other two samples have much higher moisture contents, (C06 King B (8.0%) and Ramp 5 ST4 Q2 (7.7%)). The C06 King B sample reached a maximum temperature of 74°C, before evaporative moisture heat loss overtook the heat being generated from coal oxidation. The Ramp 5 ST4 Q2 sample reached a temperature of 69°C after 63 hours of testing and can be seen to be decreasing in self-heating rate (Figure 5).

Figure 4: R70 self-heating relationship with ash content for coal samples from Meandu Mine, showing intrinsic spontaneous combustion propensity classes based on Queensland conditions

Figure 5: Adiabatic incubation test results for coal samples from Meandu Mine
The pre-oxidation of the Carey’s 2 King sample (Figure 5) was sufficient to lower the intrinsic reactivity of the coal and created a very slow self-heating response with moisture present. A similar result was obtained for the Ramp 5 ST7 Q3 sample and as such both of these samples could be classed as having no likelihood of reaching thermal runaway. The Ramp 5 ST7 Q1 sample had a very long incubation period almost three times that of the K4E ST12 AK sample, which developed into a spontaneous combustion event after approximately 4 months of being exposed. This coal could also be considered as having no likelihood of reaching thermal runaway for practical purposes in the timeframe of normal operations at the mine. In contrast however, the K4S ST3 Queen and Ramp 5 ST4 Q1 samples rate as having a high likelihood of developing into a spontaneous combustion event. Consequently, a set of well-defined management practices were used to monitor and handle this coal.

SPONTANEOUS COMBUSTION MANAGEMENT AT MEANDU MINE

As a result of the spontaneous combustion event in Meandu K4E pit ST12 and experience gained from the incubation testing of Meandu coal the following practices have been implemented as part of the PMHMP for the mine:

- All new strips are systematically channel sampled and sent to the laboratory for Incubation testing as soon as possible. When the samples arrive at the laboratory they are given priority to obtain results that can be fed back to the operations to assist with mine planning and decision making for effective and safe spontaneous combustion management.
- Based on the initial laboratory test results for the K4E ST12 AK sample and the corresponding spontaneous combustion event, coal should not be left exposed in-pit for greater than 70 days where possible, regardless of which pit or coal seam has been exposed. This provides a factor of safety of approximately 1.7. However, more recent interactive testing of coal from newly exposed strips has enabled this value to be modified to a shorter timeframe for coals that record a shorter incubation period.
- Limit coal blast areas to prevent the exposure of coal to the atmosphere for extended periods within pits that have been identified to contain low ash content coal due to its higher intrinsic reactivity.
- For coals that record a short incubation period a reduced residence time is used for the ROM stockpile. Compaction is implemented in several lifts and the batters lowered to reduce air ingress to the pile interior (Figure 6).
- Improved communications between Meandu Mine and the Tarong Power Station regarding coal deliveries to ensure risk of spontaneous combustion is managed effectively. For long term stockpiles a staged compaction process is used, which effectively turns the stockpile into the equivalent of a fractured coal pillar with low porosity and permeability. Under these circumstances the Incubation test results for these Meandu coals indicate a minimum incubation time on site of greater than 2 years. This has been verified from site experience with the Meandu coal.

In addition, a review and update of the previous Spontaneous Combustion Standard Operating Procedures has also been undertaken. The most recent “SOP” review (Document 4868-SE-SOP1027) in 2016 utilised the knowledge gained from the in-pit spontaneous combustion event and minimum incubation periods calculated from laboratory testing of all Meandu Mine coal seams. This document has been developed and implemented to ensure the safety of persons in, or near heated areas or areas with a potential for spontaneous combustion at Meandu Mine, in accordance with the Coal Mining Safety and Health Regulation 2001 Section 138.
Figure 6: Carey’s low ash “Enviro” coal stockpile, track rolled in several lifts and batters reduced to limit ingress of air in line with SOP 4868-SE-SOP1027 (“Spon Com”) guidelines

CONCLUSIONS

Coal self-heating performance for hazard assessment can be characterised using adiabatic oven incubation testing to establish the minimum incubation period (time to thermal runaway if it occurs). This practice has been applied to coal samples from Meandu Mine and the results obtained have been compared against an actual in-pit spontaneous combustion event at the mine. The closeness of the match between the laboratory result and the site performance at Meandu provides confidence in adding Meandu to a benchmarking database for future mine planning. This new benchmark result can also be used to compare coals from other mining operations for input to hazard assessment and P MHP.

The use of interactive laboratory testing as coal is exposed in-pit to obtain sound scientific data for hazard assessment planning and matching against mine site performance should be used as leading practice for industry. In view of the event at Meandu it is clear that if optimum conditions prevail spontaneous combustion can readily occur. These circumstances can be identified in advance of mining by testing the coal to evaluate the likelihood of a spontaneous combustion event under the prevailing site conditions. The data obtained can also be used as input to the auditing and improvement of current management practices.

ACKNOWLEDGMENTS

The authors would like to thank the Meandu Project for supporting this case study paper.

REFERENCES


FACTORS AFFECTING PRE-TENSION AND LOAD CARRYING CAPACITY IN ROCKBOLTS – A REVIEW OF FASTENER DESIGN

Damon Vandermaat

ABSTRACT: Recent studies into rock bolting parameters have suggested that increasing nut length has a positive influence on the pretension achieved during rockbolt installation. A literature survey on bolts and fasteners was undertaken, as well as a laboratory testing program to examine this suggestion. The testing program examined M24 nuts ranging from 1D to 1.5D (24 mm – 36 mm). These nuts were used to tension a rockbolt using an instrumented drill rig to measure torque, and a through-hole load cell to measure pretension. It was found that nut length has no impact on the pretension values achieved during rockbolt installation. The largest factor affecting pretension values was found to be the levels of friction acting between bearing surfaces. Overcoming this friction is estimated to account for 85-90% of the applied torque from the drill motor.

INTRODUCTION

Effective pre-tension in ground anchors is an important part of the ground support system. Pre-tension improves shear resistance between bedding planes, and actively works to prevent shear and vertical movement of strata layers. Correctly pre-tensioned rockbolts and cable bolts can make-or-break the effectiveness of installed ground support systems.

In order to improve the amount of pre-tension that can be achieved, it is important to understand the factors and forces that contribute to it. This paper will focus on the most ubiquitous form of ground support in underground coalmines, the rockbolt.

The majority of roof and rib bolts installed in the Australian underground coal industry are the M24 rockbolt. These bolts are typically 1.2 – 2.1 m in length, with a core diameter of 21.7 mm and metric M24 x 3 mm thread at the bottom end as shown in Figure 1. Pre-tension is applied through the procession of a 36 mm Across Flats (AF) nut along the thread, which reacts against a dome ball washer and a roof plate to impart a tensile load in the bolt.

Recent academic discussion on rockbolts has proposed an increase in nut length to improve geotechnical outcomes. An investigation into thread stability on rockbolt during pre-tension by Frith (2017), found that a 5 mm thread pitch allowed for ‘consistently higher’ pre-tension values compared to a 3 mm pitch. The conclusion from this finding was that rockbolt design should adopt a longer nut length to reduce the thread contact pressures, or move toward a 5 mm thread pitch as a means of improving thread stability during tensioning.

MECHANICS OF THREADS

The conversion of torque into pretension in a bolt is done through the procession of the nut along the thread. This procession uses the mechanical advantage of an inclined plane (helix angle) to convert the rotational energy of the drill motor into tension in the rockbolt. This helix angle is controlled by the pitch of the thread (Figure 3). The shorter or ‘fine’ the pitch the lower the helix angle and therefore, the greater the mechanical advantage.
There are a number of factors which can affect the amount of pretension that can be achieved during installation. This can include the amount of lubrication between mating parts, the thread pitch, and the quality, accuracy and tolerance class of the thread (Yan, 2014). The main contact points in the tensioning system are between the mating threads, and between the bearing face of the nut and the dome ball washer. All of these factors will affect the amount of efficiency of torque to pre-tension conversion.
An examination of the equations that govern the amount of torque required to generate a tensile force in a bolt, shows that the relationship between torque and load is independent of nut length. A commonly used equation for calculating the torque required for a specified bolt tension is given in Equation 1. This equation is a function of the bolt diameter, and a torque coefficient, which is calculated using Equation 2. The torque co-efficient considers the geometry of the thread, as well as the co-efficient of the friction between the contact surfaces (Barret, 1990). An example of calculated values for varying friction levels is presented in Table 1 and Figure 4.

\[ T = FKD_b \]

\[ K = \frac{d_m}{2d} \left( \frac{\tan \psi + \mu \sec \alpha}{1 - \mu \tan \psi \sec \alpha} \right) + 0.625\mu_c \]

Where:
- \( T \) = Torque (Nm)
- \( F \) = Tension (N)
- \( K \) = Torque Co-efficient
- \( \mu \) = Friction co-efficient threads (usually between 0.15 and 0.2 for steel-on-steel contact)
- \( \mu_c \) = Friction co-efficient bearing surface
- \( D_b \) = Bolt diameter (m)
- \( \psi \) = Thread helix angle (2.5° for an M24 x 3 thread)
- \( d_m \) = mean thread diameter (m)

Table 1: Calculation of pretension from a given torque input based on Equations 1 and 2 for a range of friction values.

<table>
<thead>
<tr>
<th>( \mu )</th>
<th>( K )</th>
<th>( T ) (Nm)</th>
<th>( F ) (T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.02</td>
<td>350.00</td>
<td>72.88</td>
</tr>
<tr>
<td>0.05</td>
<td>0.08</td>
<td>350.00</td>
<td>18.73</td>
</tr>
<tr>
<td>0.10</td>
<td>0.14</td>
<td>350.00</td>
<td>10.27</td>
</tr>
<tr>
<td>0.15</td>
<td>0.16</td>
<td>350.00</td>
<td>9.09</td>
</tr>
<tr>
<td>0.20</td>
<td>0.19</td>
<td>350.00</td>
<td>7.73</td>
</tr>
</tbody>
</table>

**Figure 4: Pre-tension achieved for various levels of friction for a 350 Nm input torque**

It can be seen that friction has a very large impact on the effectiveness of a given torque input. Realistic friction values for lubricated steel contacts are 0.15-0.2. Overcoming the effects of friction accounts for 85-90% of the applied torque. It is clear that reducing the effects of friction between mating surfaces can have a very significant impact on achievable pre-tension levels.
Deciding on thread pitch is a compromise between strength, tensioning efficiency and practicality. A coarser thread has larger pitch measurement and is less prone to stripping as a result of there being more material in the thread ‘teeth’. A finer thread has a smaller helix angle and can achieve higher bolt tension for a given torque due to a greater mechanical advantage. However, finer threads are more susceptible to corrosion, galling and contamination. Ultimately, the pitch of the thread should be kept as fine as practicable without compromising the strength and serviceability of the thread. Table 2 presents the pros and cons of fine and course threads.

<table>
<thead>
<tr>
<th>Table 2: Pros and cons of coarse and fine threads</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fine Threads</strong></td>
</tr>
<tr>
<td>Smaller helix angle – more mechanical advantage</td>
</tr>
<tr>
<td>Higher torsional strength</td>
</tr>
<tr>
<td>Higher bar strength</td>
</tr>
</tbody>
</table>

| **Coarse Threads**                           | **Cons**                                      |
| Higher stripping strength                   | Lower bar strength                            |
| Less prone to contamination                 | Lower torsional strength                      |
| Less prone to cross threading               | Larger helix angle – less mechanical advantage |

It is important to appreciate that load is not uniformly distributed on the threads along the length of the nut. The majority of loading is experienced at the face of the nut, and within the first 2 - 3 thread pitches. Up to a point, increasing the length of the nut will reduce the intensity of loading experienced at the bearing face of the nut, however there are very large diminishing returns as nut length increases (Sopwith, 1948).

An analysis was carried out based on the calculations presented by Sopwith (1948) to examine the loading characteristics on M24 x 3 threads with various nut lengths. The results of this analysis are presented in Figure 5, which presents the load intensity per unit length of thread helix. For convenience, the data is presented in terms of distance from the nut bearing face. Nut length is given in proportion to the diameter of the thread.

![Figure 5: Distribution of loading intensity along an M24 x 3 nut of varying lengths. Lengths are represented propositionally to the diameter of the thread (After Sopwith, 1948).](image-url)

*Figure 5: Distribution of loading intensity along an M24 x 3 nut of varying lengths. Lengths are represented propositionally to the diameter of the thread (After Sopwith, 1948).*
It can be seen that beyond a nut length of 1 x diameter (1D), the advantages of reducing the peak load intensity on the threads close to the bearing face is essentially zero. The distribution of load intensity on the thread beyond nut length of 1D is practically identical within the first 4-5 thread pitches. Longer nuts are able to carry very small amounts of load over their full length from pitches 6 - 15, however, the load intensity in this region is 8 – 12 times less than the peak intensity at the bearing face. The mean loading intensity also experiences diminishing returns, albeit not to the same degrees as peak load intensity. Given that pretension levels are highly dependent on the action of friction, it is unlikely that increasing the length of nut will sufficiently reduce the overall bearing pressure on the threads to meaningfully impact pretension levels.

EXPERIMENTAL TESTING

An experiment was carried out to examine the effects of nut length on pre-tension values. In this experiment 1D (24 mm), 1.25D (30 mm), 1.5D (36 mm) M24 x 3 nuts were examined. A Jennmar X-Grade rockbolt was used for the purposes of the test. A number of friction reducing agents were also used to examine the effect of friction on pre-tension. These included Teflon washers and a dry Molybdenum Disulphide coating. The tests carried out are highlighted in Table 3. The testing setup is depicted in Figure 6 and the method was as follows:

- An 1800mm rockbolt was point anchored in a 1700 mm hole (ϕ28 mm) using a 500 mm fast-set resin capsule. This produced a 900 mm un-encapsulated section at the bottom of the bolt, which represented the ‘slow’ portion of a two-speed resin capsule.
- A through-hole load-cell was placed over the thread of the bolt, and was set between two flat steel plates.
- A plate, nut and the dome ball were placed over the thread, and the nut was tensioned using a hydraulic drill rig instrumented with a digital torque sensor. Maximum torque for each test was recorded.
- The test bolt was cleaned of debris between tests, and the nut was changed after five consecutive tests.

The results of the testing are shown in Figure 7 and Table 4. The drill rig was capable of output torques of between 240-400 Nm, and pretension values ranged between 4-12T.

<table>
<thead>
<tr>
<th>Table 3: Range of test specimens.</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 mm (1D)</td>
</tr>
<tr>
<td>As Machined</td>
</tr>
<tr>
<td>Teflon Washer</td>
</tr>
<tr>
<td>Moly Coated Teflon Washer</td>
</tr>
</tbody>
</table>

Figure 6: Experimental setup highlighting major points of friction
Figure 7: Torque v's tension for 1D, 1.25D, 1.5D nut lengths with various friction reducing agents.

Table 4: Testing results

<table>
<thead>
<tr>
<th></th>
<th>Average Tension (T)</th>
<th>St Dev (T)</th>
<th>Average Torque (Nm)</th>
<th>Nm/T</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>As Machined</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1D (24mm)</td>
<td>4.5</td>
<td>0.5</td>
<td>357</td>
<td>79</td>
</tr>
<tr>
<td>1.25D (30mm)</td>
<td>4.5</td>
<td>0.4</td>
<td>360</td>
<td>80</td>
</tr>
<tr>
<td>1.5D (36mm)</td>
<td>4.9</td>
<td>0.5</td>
<td>331</td>
<td>67</td>
</tr>
<tr>
<td><strong>As Machined with Teflon Washer</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1D (24mm)</td>
<td>7.25</td>
<td>1</td>
<td>348</td>
<td>48</td>
</tr>
<tr>
<td>1.25D (30mm)</td>
<td>8</td>
<td>0.8</td>
<td>343</td>
<td>43</td>
</tr>
<tr>
<td>1.5D (36mm)</td>
<td>7.71</td>
<td>0.6</td>
<td>354</td>
<td>46</td>
</tr>
<tr>
<td><strong>Moly Coated with Teflon Washer</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.25D (30mm)</td>
<td>8.6</td>
<td>0.5</td>
<td>301</td>
<td>36</td>
</tr>
</tbody>
</table>

From the results of this testing, the impact of reducing friction can clearly be seen. Each progressive addition of a friction reducing agent increases the pretension achieved with a given input of torque. The average pretension values doubled between the as machined nut, and the molybdenum coated nut with a Teflon washer. There was no indication that increasing the length of the nut impacts the amount of pre-tension achieved. By examining the range of tests for the as machined, and As Machined with Teflon washer samples, the change in nut length made no difference to the pre-tensions achieved.

For relatively close levels of torque measurements, there is still a large scatter of pre-tension values achieved. This is likely to be due to variations in the friction co-efficient between tests. The tests were conducted over several days and it is possible that variations in temperature and humidity could impact on the levels of friction in the system. It is also expected that debris, dirt, oils, corrosion or the quality of the threads surface finish between samples may have affected the results between tests, despite efforts made to control for these variables.

CONCLUSIONS

Of all the factors that affect the amount of pre-tension achieved during rockbolt installation, friction plays the largest role. Friction can account for 85-90% of the torque required to tension a rockbolt. The main sources of friction in the rockbolt tensioning process are between the threads, and the contact face between the nut and dome ball washer. The main methods used by manufactures of rockbolts is to use a low friction washer in-between the nut
and dome ball, as well as lubricants on the thread and nut. The two types of lubricants are dry lubricants such as Molybdenum Disulphide, and wet lubricants such as grease. Wet lubricants have a higher risk to pick up dust and debris as they are transported around site, which will affect their anti-friction properties.

Nut length will have no impact on pre-tension levels. Beyond nut length of 1-1.25D, nut length will also have little impact on nut thread stability, as most of the load carried by the thread is with the first 4-5 thread pitch. Destructive testing carried out at Jennmar Australia has found that 1.25D nuts are capable of exceeding an X grade rockbolt to ultimate tensile failure - meaning that the nut is stronger than the bolt. A 3 mm thread pitch is the most widely used in the industry as it has been proven to provide the best compromise between thread strength, tension ability and resistance to contamination/corrosion.

REFERENCES

PROFILE OF SHEARED CABLE BOLTS STRAND WIRES

Guanyu Yang¹, Naj Aziz², Haleh Rasekh, Saman Khaleghparast, Xuwei Li, Jan Nemcik

ABSTRACT: For several cable bolt technology has been used for ground reinforcement in civil, mining and other construction projects. The strength properties of these cables, used as cable bolts, have been evaluated mainly by their ultimate tensile strength as this kind of test could be carried out in the field as well as in the laboratory. Only recently there has been a growing interest in cable bolt failures in shear because of documented field failure evidence. A series of single and double shear tests were carried out to study the extent shear failure of cable bolts in concrete blocks. Tests were made using both single and double shear rigs at the University of Wollongong. Various types of marketed cable bolts were tested using both types of shearing equipment. Various pertinent parameters were examined with direct influence on the failure characteristics of cable bolts were examined. This paper illustrates the strand wires failure profiles in both test methods and with particular focus being directed to the shear failures of both plain and indented cable bolts currently used in Australian mines. The nature of cable failure and the extent of sheared cable displacement affecting the profile of broken strands wires are reported to indicate the way the cable bolt has failed and its failure load.

INTRODUCTION

Cable bolting has been used world-wide as a solution for structural support and in ground reinforcement in civil, mining, tunnelling and other structure projects. The strength properties of these cables, used as cable bolts, have been evaluated mainly for their ultimate tensile strength, as this kind of test could be carried out in the field as well as in the laboratory. Only recently there has been a growing interest in cable bolt failures in shear, because of documented field failure evidence.

For the past several years, a significant knowledge has been gained on tendon load transfer mechanisms and strength characterisation mainly by pull testing (Aziz and Jalalifar, 2005, 2006, Hagan, et al., 2015), however little has been known about the cable bolt shear behaviour, since the interest in cable bolt failure in shear has been confined to small amount of work carried out based on the British Standard of shear testing (BS 7861-part 2, 2009) and the work of Craig and Aziz, 2010, and Aziz, et al.2015a). Also, no credible test results are available from the field and only pictorial evidence has recently been surfaced for both failed solid rock bolts and cable bolts. Typical signs of sheared tendons recovered from the field and a borehole view shear displacement in rock layers is shown in Figure 1, as reported by McCowan (2015), and Li (2017)

Figure 1: Sheared tendons recovered from field and a hole view of sheared rock (McCowan, 2015 and Li, 2017)

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² Professor, University of Wollongong. Email: naj@uow.edu.au Tel +61 2 4221 3449
When a cable is sheared to failure in a soft medium such as in soft rock and in weak concrete, there is little chance of the wires in the cable strand failing fully or snap in shear, instead the strand wires are likely to fail in a combination of both tensile and shear. Other influencing factors include grout strength, applied pretension load, testing method and loading condition (Aziz, et al., 2017).

This paper examines the shear failure of various cable bolts under different pretension loads using both single and double shear techniques. The extent of shear displacement has been found to play a significant role in the strand wire failure pattern as reported (Aziz, et al, 2017). Particular emphasis is focused on pictorial evaluation of the snapped strand wires.

**SHEAR TESTING OF CABLE BOLTS**

**Methodology**

Shear testing of cable bolts has, for several recent years, been undertaken by using both the single and double shear methods. A single shear test method, based on the British Standard BS 7861 Part 2 (2009), was used to determine the shear strength of cable bolts to failure (Aziz and Hawker, 2015). Aziz, (2004) undertook double shear testing of 15.2 mm diameter resin coated, seven wire strand cable bolts to examine the extent of plastic surface damage with respective to increased shearing displacement. Testing of cable bolts to full failure in shear was subsequently carried out in larger double shear machine (DS-MKII) by Aziz and Craig (2010). Further studies on cable bolt load transfer mechanism have since been undertaken with emphasis directed solely on determining the load transfer characteristics with particular reference to evaluating failure profiles of various wires in the strand.

Currently, two types of testing rigs are used for shear testing of various cable bolts, they are; Megabolt Integrated Single Shear Test Rig (MISSTR) and Double Shear Test Rig (DSTR). Prior to the construction of MISSTR, all studies in shear testing of tendons were undertaken using the University of Wollongong DSTR.

**Double shear testing method**

Two types of double shear testing methods were available for evaluating the shear characteristics of cable bolts; (a) The DSTR-MKII with opposing concrete joint faces being in contact with each other, where the resultant shearing force is a combination of the shear failure load and friction force of the sheared host medium faces, (b) A modified DSTR (MKIII), with opposing concrete joint faces not in contact with each other and the measured shear resistance force is spent on shearing the cable wires. Accordingly, The DSTR-MKIII rig as shown in Figure 2 was used in this study. The DSTR MKIII frame consists of two 300 mm length outer cubic boxes and 450 mm length middle central cuboid box with 300×300 mm² cross- sectional area. A conduit wrapped with 8 mm PVC hose, was laid horizontally along the mould to precast a rifle hole through the centre of concrete blocks. Once the concrete was poured it was left to set.

![Figure 2: Double shear test rigs (a) MKII and (b) MKIII.](image)
assembling, three concrete blocks were all mounted on the horizontal steel base and held together using a truss system around the double shear assembly as shown in Figure 1 b. The truss system consisted of four 1100 mm length steel braces connected between two 30 mm thickness side steel plates. The brace system impedes subjecting lateral axial load on concrete blocks during shearing. When assembled, gaps of almost 5 mm were left between concrete blocks, thus the adjacent sheared concrete faces are kept apart thus eliminating the contact between the sheared faces and hence no friction force. Next the cable bolt was inserted into the central axial hole and was followed by mounting 100 t load cell on each protruding side of cable in the assembled concrete blocks and tensioned to the predetermined axial pretension load, using a “Blue Healer” tensioner. Tensioning of the cable was retained by the barrel and wedge retainers. This was followed by the injection of the grout in the central concrete blocks hole for bolt encapsulation. Grouting of the cable in the concrete block was achieved via 20 mm diameter holes cast on top of each concrete block. Once the cable was pretensioned, grout mortar was injected the space between central and cable strand from the vertical pre-cast hole in the top of each concrete blocks. After seven days of grout/resin curing time, the double shear assembly was then placed on the carrier base frame consisting of a parallel pair of rail track sections welded to a 35 mm thick steel plate. The outer side 300 mm cube blocks of the double shear apparatus was mounted on 100 mm steel blocks, leaving the central 450 mm long block free to be vertically sheared down using a 500 t capacity hydraulic universal testing machine at the rate of 1 mm/min for the maximum 100 mm vertical displacement. A hydraulic universal testing machine with a capacity of 500 t was used to compress middle block for shearing cable strand at the rate of 1 mm/min for the maximum 100 mm vertical displacement.

**Single shear test**

To replicate as closely as possible the field conditions for the installed cable bolts, the Megabolt Integrated Single Shear Test Rig (MISSTR) shown in Figure 3 was used to evaluate the behaviour of cable strand in shear. Based on the principle of British Standard 7862-part 2 (2009) the whole length of the concrete cylinder used in MISSTR is 3600 mm (1800 mm on each side) with 250 mm outer diameter hole diameter of 28-55 mm in diameter. The diameter of the central axial hole in the concrete was dependent on the diameter of tested cable bolt.

The MISSTR is a horizontally aligned integrated system consisting of a shearing rig and an integrated 120 t capacity compression machine. The 3.6 m long concrete shearing cylinder consists of two sections, each containing 1.8 m long concrete cylinders. The concrete cylinders are covered by steel clamps, which provide confinement during the shearing process. Either a hand pump or a power pack of a suitable capacity applied the hydraulic pressure for compression machine legs. The pressure in the manifold was monitored with a digital pressure transducer (Type Measure X, range 0- 800 Bar) in conjunction with an analogue pressure gauge (0-700 bar). The rate of loading was applied manually, which was not constant, however the aim was to apply a constant load at the rate of around 1 mm/min (0.018 mm/sec), in line with BS7861-2 standards. The displacement at the shearing plane
was measured using a Linear Variable Differential Transformer (LVDT) as shown in Figure 3 a. Two other LVDTs were also mounted on the cable ends to enable monitoring of cable debonding. A data taker recorder was used to collect data during the tests.

When preparing, two 1800 mm concrete cylinders glued by two 900 mm cylinders were butted together in a specially built tensioning frame. The cable bolt is then inserted through the centre rifled hole of the concrete cylinder. The cable bolt was pre-tensioned. The whole concrete cylinder loaded frame with cylinder was then tilted for 65 degree and the grout was pumped from the bottom up to the hole to remove any air bubbles remaining inside the grout annulus area and to ensure full cable encapsulation. Stratabinder HS grout was used to encapsulate all tested cables in this programme of study. The strength properties of the grout have been reported by Majoor et al. (2017), and Mirza, et al. (2016).

After a grout curing period, each concrete sample with encapsulated cable bolt was disassembled from the frame and lifted out to be mounted on to the shearing rig. Once the concrete cylinders was correctly placed in the shearing, steel clamps were placed around the concrete blocks to provide a confining pressure to the sample, this accurately replicating the in situ conditions. When sheared one side of 1.8 m of the 3.6 m concrete column remains fixed on the rig, while the other half is subjected to shearing. Two LVDTs were installed on each end of the cable strand to monitor the point displacement during shearing. The load of shearing face was recorded by data taker and the displacement of cable ends and sheared cable strand wires were monitored by LVDTs, which were all logged by computer.

INFLUENCE OF STRAND WIRE PROFILING

To study the influence of cable strand wire profiling, various cable bolts that are available in the market were tested using both single and double shear rigs. All tests were carried out in a 40 MPa concrete medium. Both plain and indented cable bolt strand wires failure profiling in both test methods are pictorially illustrated in Figures 4 and 6. In double shear testing two types of cable bolts, Sumo and Megabolt cable bolts, were considered, while more cable bolts were tested using the Megabolt single shear test rig. The focus of attention was to examine the influence surface profiling and bulbing on cable bolt failure mode and cable strand wires failure modes. In particular the role of strand wires roughness was evaluated with respect to cable bolt strand wires failure pattern. Table 1 lists properties of both Sumo and Megabolt strand wire cable bolts plain and indented surfaces. The Megabolt cables included MW9 spiral and MW10 Plain cables. MW9 is a nine wire strand cable while MW10 Strand has 10 wires. Sumo cable bolts have 9 wire strands. Table 2 shows the comparative shear test results of SUMO cable bolt strand of smooth and indented wires.

Table 1: Properties of SUMO cable strand and MW cable strand from manufactory

<table>
<thead>
<tr>
<th>Cable bolt</th>
<th>Indented SUMO</th>
<th>Plain SUMO</th>
<th>Spiral MW9</th>
<th>Plain MW10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>28 mm</td>
<td>26 mm</td>
<td>31 mm</td>
<td>31 mm</td>
</tr>
<tr>
<td>Capacity</td>
<td>630 kN</td>
<td>650 kN</td>
<td>620 kN</td>
<td>700 kN</td>
</tr>
</tbody>
</table>

Table 2: Single and double shear test results of plain and indented SUMO cable bolt strand wires at 0 and 15 t pretension load

<table>
<thead>
<tr>
<th>method</th>
<th>double shear test (no joints surface contact)</th>
<th>single shear test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st (kN)</td>
<td>0t (kN)</td>
</tr>
<tr>
<td>item</td>
<td>Disp. (mm)</td>
<td>Load (kN) (single face)</td>
</tr>
<tr>
<td>Plain cable</td>
<td>88.2</td>
<td>852 (426)</td>
</tr>
<tr>
<td>Indented cable</td>
<td>85.7</td>
<td>767 (384)</td>
</tr>
</tbody>
</table>

Profiles of failed wire surface

**Double shear tests:** Figure 4 shows the profiles of both SUMO and Megabolt MW9 and MW10 cable bolts tested in double shear rig.
Because of the ineffective concrete confinement in rectangular and cubical double shear rig, the radial cracking of the concrete blocks, as shown in Figure 5 generated conditions that caused increased sheared cable vertical displacement. The increased displacement contributed to increased cable failure loads, significantly greater than that would have occurred with effective confinement and no radial cracks. The radial cracking of the concrete enables the sheared cables to bend excessively along the weak cracked zone making the cable bent zone behave as though being pulled apart leading to failure by pulling rather than shear and hence increased failure load. As can be seen from Figure 4 and profiles of cut strand wires, it is obvious that most wires have failed in tension and a combination of shear and tension.

Also, unconfined concrete host medium may end up being radially cracked during the shear process, making the cable shear displacement greater, causing the cable to snap or fail as if it is being pulled to failure rather than being sheared and the resulting failure being greater, at around 70%, than would normally be the case.

Figure 4: Strand wires failure profiles in Sumo and Megabolt cables tested in DS TR-MKIII Equipment. Both left and right shear failure faces are shown

Figure 5: Double shear post-test concrete blocks crack
**Single shear test:** A total of 19 cable bolts were tested, which included: (a) nine Megabolts of six MW10 plain and three MW9 spiral, (b) two SUMO Plain and two SUMO Indented, (c) others. Table 2 lists 16 cable bolts tested at predetermined pretension loads of zero and 15 tonnes. Figure 6 shows the profiles of 12 tested cables tested using MISSTR. The focus in this paper was to compare shear profiles of two types of cable bolts, namely Sumo and Megabolt MW with respect to the type of testing methods (single versus double shear methods).

Table 2: single shear test results of 19 cable bolts.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product Name</th>
<th>Cable Dia. (mm)</th>
<th>UTS (t)</th>
<th>Cable geometry</th>
<th>Pre-tension load (t)</th>
<th>Peak Shear load (t)</th>
<th>Shear disp.</th>
<th>Cable debonding</th>
<th>Peak shear load/UTS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MW 10-P</td>
<td>31</td>
<td>70</td>
<td>Un-bulbed</td>
<td>15</td>
<td>68.34</td>
<td>68.24</td>
<td>Yes</td>
<td>97.6</td>
</tr>
<tr>
<td>2</td>
<td>MW 10-P</td>
<td>31</td>
<td>70</td>
<td>6 bulbs</td>
<td>0</td>
<td>63.84</td>
<td>62.57</td>
<td>Yes</td>
<td>91.1</td>
</tr>
<tr>
<td>3</td>
<td>MW 10-P</td>
<td>31</td>
<td>70</td>
<td>6 bulbs</td>
<td>15</td>
<td>60.39</td>
<td>56</td>
<td>Yes</td>
<td>86.3</td>
</tr>
<tr>
<td>4</td>
<td>MW9-S</td>
<td>31</td>
<td>62</td>
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**RESULTS AND ANALYSIS**

Because of the lack of effective confinement in double shear equipment, the failure profiles of most snapped wired in various cables were mostly in tension, tensile/shear and no failures. No failure often occurred on one side of the double shearing because the individual wires in the strand began to debond causing wires to be pulled out when sheared down further with excessive displacement. As a result the recorded shear strength of the cable was significantly higher than those noted from single shear tests, Hence no realistic conclusions can be made on DS test wires failure.

Also and although no direct comparative analysis can be made between single and double shear methods with respect to cable debonding, nevertheless the cable debonding significance is appreciated when testing cables with long encapsulation length, as in single shear test. This has provided a significant input on the cable bolt performance with regard to cable strand wires failure due to wire surface roughness, cable encapsulation length, debonding and the extent of cable shear displacement at the shear section zone. Plain MW10 was found to be superior in terms of resisting shear load. This is because the increase in number of wires in the strand to ten and had a tensile strength of 70 tonne. From the single shear testing it was found that:

- Plain wired strand cables were debonded for the tested cable encapsulation length of 1.8 m. Some debonded cables did not fail or snap. No debonding occurred in indented cable
<table>
<thead>
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<th>Figure 6: strand wires failure patterns for all tested cables</th>
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<tr>
<td>1</td>
</tr>
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<td>2</td>
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</table>

Yellow circles represent tensile failure, and blue circles represent the combination of tensile and shear failure.
strands wires for the same length of cable encapsulation. Secura cable with almost 40% spiral wires did not debond.

- The debonded cables, which were mainly plane wired, resulted in an increase in displacement at peak shear load. Where a cable is debonded, the shear displacement of between 60 - 120 mm made the shared cable strand section behaves as if it was being pulled apart to failure rather than being sheared. As a result wires tend to fail in tension or a combination of shear and tension with the failure in tension being predominant as can be seen in Figure 7. Thus it is reasonable to suggest that increased displacement has influenced the nature of sheared wires failure in the strand. Generally, wires fail closer to shear/tensile state in comparison to failure in tension.
- Increased cable displacement causes cable wires to fail mostly in tension. The cable failure in tension also occurs in double shear testing as a result of concrete medium cracking axially as shown in Figure 5, thus resulting in excessive bending and stretching of the cable section in the vicinity of joint faces, and shear displacement travel in double shear tests was mostly in excess of 70 mm. Often the failure loads were in excess of the failure loads obtained from single shear testing. Thus the majority of wire failures tend to be in tension failures with a small number of wires failing in a combination of tension and shear.
- No debonding was observed in spiral and indented cable bolts because of the influence of increased interlocking in the cable/grout interface. Accordingly failure modes in the cable bolt wires are the combination of tensile and tensile/shear failures. No strand wire fails in pure shear unless the wire is fully guillotined.
- In general, the failed strand peak shear load was lower with increased pretension load in a bonded condition in shear. Higher pretension load causes the cable to stiffen and fail with lower vertical shear displacement.
- Pure shear in cable wires occurs when the cable is guillotined, with wires being squeezed and with lower shear load as reported by McTyre and Evans (2017). In double shear testing it is impossible to observe cable debonding because of barrel and wedge influence.
- It appears that reducing the rate of loading provided consistent results. It is accepted that the rate of loading of less than 4 mm /min is a reasonable rate for testing.

With single shear testing facility, the preparation of up to six single shear test samples was possible at the same time, hence the grouting of cables was left to cure for a desired cure time and they were all tested over a short time span. All tested samples were fully and effectively confined with steel clamps leading to consistency in results. However, the methodology is laborious and requires heavy lifting gear. Also, prolonging the sample testing duration may allow studies on the influence of host concrete medium and grout age to be investigated effectively without sacrificing any monitoring equipment attached on the prepared samples. This was not the case with the double shear testing method, due to the fixed barrel and wedge anchors; it was only possible to encapsulate one cable at a time with load cells being locked up on the cable’s ends. As a result tests were made in shorter grout age of around 10 days.

The next programme of double shear testing will be undertaken in short cylindrical concrete blocks with effective steel clamps that will prevent concrete radial cracking for improved results.

CONCLUSIONS

The rectangular steel clamps on prismatic DSTR cannot provide effective confinement to the concrete medium. Thus, the unconfined concrete host medium may prematurely crack during
the shearing process, with increased shear load displacement, thus causing the cable to fully snap or fail as if it is being pulled to failure rather it is being sheared.

Because of cable debonding, not all debonded plain cable strand wires failed or snapped in single shear testing for the given cable encapsulation length of 1.8 m. Increased displacement influenced the nature of sheared strand wires. Generally wires failed closer to the shear/tensile state in comparison with failure in tension. Increased cable displacement causes cable wires to fail mostly in tension.

In general the failed strand peak shear load was lower with increased pretension load in the un-debonded condition. The peak shear load failure of un-debonded cable was lower with increased pretension load. Higher pretension load causes the cable to stiffen and fail with lower vertical shear displacement.

The use of steel clamps in double shear method with cylindrically shaped concrete would prevent concrete crack radially and may provide a sound way of testing cables for shear similar to single shear tests.

Figure 7: Cross sectional view of Cable bolt Unbulbed, Plain MW10, 15 t pretensioned and debonded. Note the extent of wire failure in tension of Cone and Cup. The failure load of 68.2 t is almost equal the cable axial load capacity of 70 t. The shear load displacement at failure was 68.2 mm.

ACKNOWLEDGEMENTS

The single shear apparatus (MISSTR) used in this study was developed by Megabolt Australia, and was made available to researchers to undertake shear characterisation of various cable bolts used for ground reinforcement in Australian mines. The project was
funded by ACARP (project C24012), with in kind support from various bolt manufacturers and the University of Wollongong. Accordingly, the research team wish to thank Megabolt Australia for the loan of the MISSRT and making available Mr Owen Rink and occasionally Mr Ron McKenzie to assist in training and preparation of samples, thus maintaining consistency in testing. Special thanks goes to Alan Grant, Colin Devenish, Duncan Best and Travis Marshal, the technical staff of the School of Civil, Mining and Environmental Engineering, Faculty of Engineering and Information Sciences, the University of Wollongong, for their technical expertise support in bringing this project to a meaningful conclusion. Various types of cables used in this study were provided by bolting companies including Megabolt Australia, Jennmar Australia, and Minova /Orica. Orica / Minova also supplied Stratabinder grout for this study.

REFERENCES


DEVELOPMENT, TRIALS AND TESTING OF A TWO COMPONENT RAPID SET CEMENT GROUTING SYSTEM

Tom Meikle¹, Robert Hawker², Colin Grubb, Stephen Tadolini, Peter Mills

ABSTRACT: In unfavorable strata conditions, it is often necessary to install additional long tendon support as part of a primary support cycle. Although these support systems provide the required additional stability to the roadway, installation does have a significant effect on the rates of development. To overcome the slow cure time of Portland cement type grouts, the use of two component resin systems such as Polyurethanes and Urea Silicates have become favored but concerns over creep properties has led to the development of a new two component grouting medium which displays the properties of traditional cable grouting materials with the rapid reaction times of pumpable resin systems. This paper describes product testing, product application - surface trials, underground trials and full-scale applications of the Tekthix system in a dynamic underground environment.

INTRODUCTION

Single component cement grouts have been used to anchor long tendon supports for many years. They range from basic high and low slump Portland cement-based slow setting grouts to fast setting single component grouts. However, current cement grouting systems (grout and equipment) have some disadvantages:

1. Low slump grouts require pumps that can pump a thick high viscosity grout, so choice of pump is limited.
2. Pumping distance is limited with current cable bolt grout pumps; operators often tend to add additional water to allow pumping long distance, which adversely impacts the quality and strength of the material.
3. Fast setting grouts tend to cause issues with pump and line blockages due to reduced working life of these single component materials.

The industry required a fast setting grout that could be pumped further than current single component grouts, whilst still having high values for UCS, Young’s Modulus, Flexural and Adhesive strengths. Any change to current ground control systems, requires intensive testing to take place to ensure that the support system used has not been compromised by any change Tekthix has been developed to meet these requirements.

TEKTHIX TWO-COMPONENT CEMENT GROUT

Tekthix can best be described in the following terms:

- A two component cement grout
- Used for grouting of any long tendon support system
- Individual components mixed at water: powder (w:p) of 0.35:1
- Mixed Part A with Part B in a volume ratio of 1:1
- Capable of being pumped in excess of 200 m to point of application
- Mixed through static mixer at site of application
- Reacts quickly to form thick paste for “top down” grouting within seconds of mixing and sets within minutes to provide early load transfer between steel tendon and strata.

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² Technology Manager. Email: robert.hawker@minovaint.co

University of Wollongong, February 2018
LABORATORY TESTING

The following laboratory tests were carried out on the product:

- Uniaxial Compressive Strength
- Flexural Strength
- Tensile Strength
- Young’s Modulus
- Creep Testing
- Adhesive Strength
- Viscosity of single components
- Double Embedment Pull Testing

TESTING REGIME

Mechanical properties

The Tekthix individual components were mixed at the corresponding water to powder ratio of 0.34-0.35:1 at room temperature. Following this the two mixtures were mixed together at a 1:1 ratio by volume and cast into various cubes and cylinder moulds for testing. The samples were left to cure at room temperature for the corresponding period of time then tested using either 500kN compression machine also a Shimadzu 50 kN compression machine.

Uniaxial Compressive Strength (UCS)

The UCS of Tekthix ranged between 25 MPa at one hour and 75 MPa at 28 days. By comparison, currently used Portland based cement grouts have UCS values in the range of 30 MPa at 24 hours and 70 MPa after 28 days – Figure 1 and Figure 2.

Figure 1 shows Tekthix UCS at 0.34:1 and 0.35:1 Water/Solids ratio. Figure 2 shows the comparison of UCS with two other single component grouts from 1 hour to 28 days.
Flexural Strength

Figure 3 shows Flexural strength growth at 1hr, 24hrs and 7 days, Figure 4 shows sample being tested in the Shimadzu 50 kN compression machine. The Flexural Strength of Tekthix ranged between 2 MPa at 1 hour and 8 MPa at 7 days.

Tensile Strength

The Tensile Strength of Tekthix ranged between 3.5 MPa at 1 hour and 5.5 MPa at 7 days. Figure 5 shows results for, 1 hour, 1 day and 7 days. Figures 6 and 7 show the testing process using the Shimadzu 50kN compression machine.
Figure 6: Axial tensile testing of sample

Figure 7: Post-test split sample

Young’s Modulus

Figure 8 shows Young’s Modulus after 1hr, 24hrs and 7 days. Young’s Modulus values ranged from 3000 MPa after 1 hour to 4600 MPa after 7 days, by comparison, two component Urea Silicate grouts or Polyurethanes have Young’s Modulus values below 1000 MPa.

Figure 8: Young’s Modulus of Elasticity of Tekthix grout at various curing times/ages

Creep (BS7861)

Samples of dimension 12.5 mm x 12.5 mm x 50 mm where subjected to constant load of 5 kN and the change in strain was measured between 30 seconds and 15 minutes. BS7861 calls for strain % of less than 0.12% to comply. Figure 9 shows percentage creep. 
Adhesive Strength

To evaluate bond strength of Tekthix in comparison to Geoflex Urea Silicate Resin, two sandstone blocks of dimensions 50 mm x 50 mm x 100 mm were bonded together with a bond width of 3 mm to 5 mm and left to cure for two hours. Figure 1 shows the average grout UCS after one hour of casting. The bonded blocks were then subjected to a flexural strength test and bond strength calculated.

Table 1: Average Stress N/mm²

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<th>Time after casting</th>
<th>Tekthix</th>
<th>Urea Silicate</th>
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<tr>
<td>1 hour</td>
<td>3.17 N/mm²</td>
<td>1.64 N/mm²</td>
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Component Viscosity

As the viscosity of both components increases with time, it was important to understand individual pot life of both components. It was evident that if either component was left standing after initial mixing, that the viscosity would slowly increase to such an extent that it would have an effect on the pumpability of that component which could lead to pumping difficulties. The low viscosity of the individual Tekthix components enable pumping distances in excess of 200 m, by comparison to standard Portland cement grouts have a limitation of approximately 30 m with current pumps. Figure 10 shows rate of increase in Viscosity at 25°C with product standing and mixing.

Double Embedment Pull Testing (DEPT)

This test was carried out using an Instron Hydraulic Universal Testing Machine. The objective of this test was to compare the load transfer performance of Tekthix and a common Industry used single component Portland cement grout, Stratabinder HS (SBHS) Tekthix testing was carried out after 4-5 hours, single component grout after 24 hours.

Grout details

The Tekthix grout used in the 320 mm steel tube sections was tested at 4 hours and gave a UCS of 35MPa. The Stratabinder HS grout used in the comparison 320 mm tube sections was mixed at 0.4:1 water to powder ratio, which when tested at 24 hrs gave a UCS of 20MPa.

Laboratory testing overview

The main aim of the laboratory testing was to determine the mechanical properties of Tekthix compared to current Portland cement single component grouts. In all aspects, Tekthix proved to have higher UCS, Adhesive Strength. Comparison of results achieved using the Double Embedment Pull Test (DEPT) test method has shown that Tekthix grout at 4-5 hours cure...
time) achieved double the peak loads over Stratabinder HS at 24 hours cure time when either the Secura HGC or Megabolt MW9 cable are used.

![Figure 10: Tekthix A and B Viscosity changes at 25 °C, with respect to product standing and mixing](image)

The results also showed that load stiffness between 50-150 KN is 40% higher on average for Tekthix (37 KN/mm) over Stratabinder HS (26 KN/mm) at the tested cure times. Total work energy to 100 mm displacement is also as expected over 40% higher on average for the six tests for Tekthix (25 kJ) over Stratabinder HS (10 kJ). These results clearly indicate the higher load support benefits for Tekthix at an early age over Portland cement type grouts such as Stratabinder HS even with 24 hours curing (cure) period.

Figure 11 shows Cable set up for DEPT, Figures 12 and 13 show cable after DEPT Testing and Figures 14 and 15 show results of DEPT on both the Secura and MW9 Cables using Tekthix.

![Figure 11: Pull testing set up](image)
Figure 12: Post-test pulled out cable

Figure 13: Close-up view of pulled out cable

Figure 14: Load-displacement of Secura Cable bolt using different grouts.

Figure 15: Load-displacement of MW9 cable bolts with different grouts.
PRODUCT APPLICATION – SURFACE TRIALS

The following surface trials were carried out prior to any grouting trials.

Product recirculation through pump

The A and B components were mixed at 0.35:1 water to solids ratio and re-circulated through the pump for 4 hours to confirm:

- Pot life of individual components
- Separation of product
- Piston seal wear/damage
- Product build up inside pistons.

Re-circulation results

- Changes in viscosity caused by recirculating through the pump were no different from those found when mixing the product.
- There was no sign of the solid components separating from the water
- Delivery rate through the pistons remained unchanged and on target
- There was no visible sign of seal wear or damage
- There was no buildup of material in the piston or around the seals.

Maximum pumping distance and optimum hose diameter

150 m of DN20 hose was fitted to the outlets of each component cylinder. The pump was set at 60 cycles per minute (60 upward and 60 downward strokes), which delivered product at a rate of 12 liters/Min per side (24 liters per minute combined). Pressure gauges set in line showed zero pressure. A further test was carried out with 250 m of DN20 hose, which showed similar results.

Pump stall pressure

Pressure gauges and shut off valves were set in line to each cylinder. The pump commenced stroking at 60 cycles per minute and each valve slowly closed. When fully closed, output pressure. Figure was 120 bar.

Mixing nozzle configuration

The aim of these trials was to determine which mixing attachment both mixed the components correctly and achieved minimal backpressure. After several trials, a nozzle that consisted of DN20 fittings (Figure 16) and a 12 module x 16 mm static mixer (Figure 17) provided the best outcomes.
Grouting trials

To simulate a cable inside a borehole, 6 m x 38 mm hollow cables were inserted into 40 mm conduit to confirm that the system could achieve full encapsulation Figure 18. 50 m of DN20 hoses were installed onto the pump. Once the grout was allowed to set, cables were cut into 2m sections to confirm full encapsulation Figure 19. This process was repeated several times with water temperature ranging from 24°C to 30°C.

Grouting outcomes

The system proved to be capable of pumping Tekthix through 50 m of DN20 hose and achieves full encapsulation each time. Figure 18 shows full encapsulation of 6 m cables. Figure 19 shows cable cross section.

UNDERGROUND TRIALS

The first underground trial was carried out at a New South Wales Coal Mine. A series of hollow cables were successfully grouted without any issue. Grouting time ranged from 3 – 4 minutes. Grout was seen issuing from the telltale hole in the plate confirming full encapsulation had been achieved. Pump strokes were counted to confirm the volume of grout pumped into each cable was equal to or greater than the theoretical volume of annulus and grout tube inside the cable. Figure 20 shows grout lance attachment, Figure 21 shows grout tell tale.

Having reviewed the results of the initial trail, and additional product testing information provided by Minova, the mine decided to fully implement Tekthix in the widening of the next
longwall installation road. The first pass of the installation road at 5.5 m wide is followed up by a 2nd pass of 2.5 m wide, to achieve a total width of approximately 7 m. The second pass consists of cutting to full width for approximately 20 linear meters. In order to provide additional stability to the wide span, a series of fully grouted, 6 m and 8 m long tendon cables are installed. Using standard single component grout, the grout is allowed to set for a minimum period of 24 hours before cutting of the 2nd pass recommences.

With the rapid set time of Tekthix, the wait time was reduced to less than 4 hours. A total of 350 cables were installed and grouted with Tekthix reducing the strip out time substantially.

CONCLUSIONS

The Mechanical properties of Tekthix two-component grout are equivalent to, or greater than current single component grouts. In addition, Tekthix also offers the additional advantage:

- Rapid strength gain allowing in cycle bolting and substantial reduction in cycle times,
- Higher Load Transfer performance than single component grouts,
- System can be adapted to be used with any support system that uses cement grouts,
- Pumping distance way in excess of current single component cement grouts.

To date, over 1000 cables have been grouted using Tekthix, and at date of publishing, three more large campaigns are planned in both New South Wales and in Queensland coal mines.

Further development in the pumping and mixing system is ongoing in order to reduce mixing time of grout and also to explore pumping this type of grout from a surface installation to underground location.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the effort of the Minova Australia/Asia support teams in the development of this innovative solution.
COMPARISON OF THE PERFORMANCE OF RESIN AND CEMENTITIOUS GROUTING MEDIA FOR CABLE BOLTS

Edward Pullan¹, Danqi Li², Paul C Hagan³

ABSTRACT: Cable bolting systems are an integral part of ground support designed to improve the stability of underground excavations. As geotechnical conditions vary, each component in the cable bolt system must be optimized to maximise the efficiency of the system. This study examined the performance of cable bolts with two different types of grouting agent – a standard cementitious material and a resin-based grout; in two strengths of confining material; and, in two different borehole diameters. Performance was quantified in terms of peak load, residual load and stiffness. The UNSW modified Laboratory Short Encapsulation Pull-out Test (LSEPT) facility was employed where an axial load was applied to a high capacity modified cable bolt. Results of the study indicated significant differences in the performance of the cable bolt between being grouted in strong and weak materials with the former resulting in the highest average peak loads of 406 kN and 397 kN respectively for the cement and resin grouts respectively indicating both grouts were just as effective in load transfer. The average peak load in the weak material was about 24% less at around 315 kN and in one case 207 kN with cement grout in the standard borehole diameter. In general, peak load was slightly higher when grouted in the standard borehole diameter in strong material but this trend was reversed in the weak material. Interestingly in terms of residual load, or the load bearing capacity after 90 mm displacement, the reduction from the peak load was much less at just 35% in the weak material to 203 kN whereas the reduction in the strong material was 63% to 145 kN. Little difference was observed in the stiffness between all test scenarios.

INTRODUCTION

Ground support is an essential element in ensuring the stability of underground excavations. Cable bolts are one example of ground support developed originally during the 1940’s in North America that began to be used in Australia during the 1960’s (Bouteldja, 2000). Despite extensive research being undertaken on the effectiveness of cable bolt systems, most recently by Chen, Hagan and Saydam (2016) roof failures still continue. For example, Mark, Molinda and Doliner (2001) reported that each year in the U.S.A., some 1,500 reportable non-injury roof falls occur.

This paper aims to improve understanding of the performance of cable bolts in different conditions. The testing program was undertaken using the UNSW modified Laboratory Short Encapsulation Pull Test (LSEPT) facility which applies an axial load to the cable bolt with the aim of determining the effect of changes in grout material, borehole diameter and strength of rock on the peak load, residual load and initial stiffness of a cable bolting system. This was achieved by comparing the performance of a high capacity modified cable bolt grouted into a confining material using a Jennmar ‘standard single speed’ oil-based resin and a Minova ‘Stratabinder HS’ cementitious grout. A Megabolt MW9 spiral wire cable bolt was grouted in weak and strong confining media having UCS values of approximately 15 and 50 MPa respectively and, in standard (42 mm) and oversized (52 mm) diameter boreholes.

TEST PROCEDURE

The UNSW modified LSEPT facility is based on the British Standard BS7861-2:2009 that has been modified to overcome deficiencies when testing high capacity modified bulbed cable bolts on the market and used in the Australian underground coal industry. In the modified test, a cable bolt is embedded in a confining medium with an external diameter and length of 300 mm and 450 mm respectively, these being nearly twice the dimensions specified in the

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BS test design. Earlier work by Chen, Hagan and Saydam (2017) found the pull-out load of a high capacity modified cable bolt varied with the diameter of the confining medium in which the cable bolt is embedded up to 300 mm beyond which there was little further change in pull-out load as shown in Figure 1.

![Figure 1: The effect of sample diameter on peak load of a cable bolt (Ur-Rahman and Hagan, 2015).](image)

The test program consisted of eight different combinations of strength of confining medium (weak and strong); grout type (resin and cement); and, borehole diameter (standard and oversized) as detailed in Table 1. The program allowed for each combination of test parameters to be replicated five times and, as per the British Standard, the three best results have been reported.

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</tbody>
</table>

The fully assembled test arrangement shown in Figure 2 comprised a cable bolt embedded in a cylindrical cement-based confining medium. A bearing plate was placed on top of the confining medium with a centre hole that allowed the cable bolt to extend up through a hollow hydraulic actuator and load cell with a barrel and wedge assembly attached to the end of the cable bolt. A hydraulic actuator was used to apply the load to the cable bolt with a pull-out displacement of 100 mm. The load cell and an LVDT were used to measure the applied load and the resultant displacement of the cable bolt. The confining medium was placed within a split-steel cylinder which was bolted together and the bolts pre-tensioned to 40 N·m prior to testing. Changes in the anchorage performance of the cable bolt were assessed in terms of the average peak load attained during pull-out, the initial stiffness of the cable bolt up to the point of peak load and the residual post-peak load that could be sustained by the cable bolt after 90 mm displacement.
Figure 2: The assembled modified LSEPT testing facility and measurement instrumentation (left); split-steel confining cylinder (upper centre); bearing plate (lower centre); and, confining medium showing 100 mm of extruded cable bolt after completion of a test (right).

STRENGTH OF TEST MATERIALS

Strength tests were undertaken of the two batches of confining medium used in the program; oil and water-based resin products; and, five mixes of Stratabinder, the cement-based material used as the cement grout, at varying water to cement ratios. The results of the strength tests are shown in Figure 3.

Figure 3: Results of strength tests on confining media, resin grout and cement grout.

The strength tests showed there was a three-fold difference in strength between the weak and strong confining media (15.7 vs. 49.2 MPa respectively) and, a three-fold difference between the water-based resin and oil-based resin (20.4 vs. 61.9 MPa). As it was found that the water-based resin had a very low strength, only the oil-based material was used as the resin grout material. A water to cement ratio of 0.49:1 was used for the cement grout being equivalent to the strength of the oil-based resin.
LOAD TRANSFER BEHAVIOUR IN A WEAK CONFINING MEDIUM

Standard borehole diameter

As indicated in Figure 4, there was on the whole consistent repeatable performance observed with the resin grout used to embed the cable bolt in the standard manufacturer’s recommended borehole diameter of 42 mm and in the 15.7 MPa weak confining medium both with regard to the peak pull-out load and residual load. With respect to the resin grout, the peak load varied between 279 and 333 kN, with an average value of 306 kN. There was somewhat slightly more variability in the peak load with use of the cement grout varying between 165 and 265 kN with an average peak load of 207 kN. The corresponding displacement to peak load was similar for both grout types at between varied between 20 and 30 mm.

Figure 4: Load transfer behaviour of a cable bolt with resin grout (left) and cement grout (right) with a standard borehole diameter in a weak confining medium.

There was a marked difference in the behaviour between the two grout types in nearly all tests. In the case of the cement grout tests, there was very often a sudden reduction in the post-peak load bearing capacity approaching 60% in many instances. Also, the “slip-lock” phenomenon was observed in the post-peak region more often with the cement grout.
Table 2: Comparison of test results between resin and cement grout in weak confining medium.

<table>
<thead>
<tr>
<th>Borehole diameter</th>
<th>Grout type</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>Residual strength at 90 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard (42 mm)</td>
<td>Resin</td>
<td>306</td>
<td>9.2</td>
<td>224</td>
</tr>
<tr>
<td>Standard (42 mm)</td>
<td>Cement</td>
<td>207</td>
<td>8.3</td>
<td>121</td>
</tr>
<tr>
<td>Oversized (52 mm)</td>
<td>Resin</td>
<td>364</td>
<td>15.2</td>
<td>257</td>
</tr>
<tr>
<td>Oversized (52 mm)</td>
<td>Cement</td>
<td>358</td>
<td>9.9</td>
<td>138</td>
</tr>
</tbody>
</table>

Note: Residual load is defined as the load that can be sustained by a cable bolt after 90 mm displacement.

As indicated in Table 2, there was much less of a reduction in the residual load with the resin grout compared to the cement grout indicating the former tended to maintain its integral shape whereas the cement grout was found to crumble offering less resistance as it was drawn through the confining medium. The stiffness was in the majority of test results of the same order for the resin and cement grout averaging 10.5 kN/mm. There was a marked difference observed in the failure pattern between the two grout types in the weak confining medium. As shown in Figure 5, failure with the resin grout tended to occur at the grout to rock interface whereas with the cement grout, failure occurred at the cable to grout interface.

![Figure 5: Typical failure in the weak confining medium at the grout/rock interface with the resin grout (left) and failure at the cable/grout interface with a cement grout (right).](image)

Oversize borehole diameter

The 10 mm larger borehole diameter in the weak confining medium resulted in an increase in performance with both grout types. In the case of the resin grout, there was an improvement in anchorage performance with much higher average peak load, residual load and stiffness than in the standard borehole diameter, these being 364 kN, 257 kN and 15.2 kN/mm respectively as shown in Table 2.

While peak load was fairly consistent between the tests, there was much more variability in the residual load with the resin grout than that observed in the standard borehole diameter as can be seen in Figure 6. There was in general more consistent behaviour with use of the cement grout in the oversize borehole and a substantial increase in average peak load from 207 kN in the standard diameter to 358 kN in the oversize borehole. However, there was little substantial change observed in residual load and stiffness.
LOAD TRANSFER BEHAVIOUR IN A STRONG CONFINING MEDIUM

Standard borehole diameter

In all instances in the strong confining medium, failure occurred at the cable to grout interface. Overall, embedment in the stronger confining medium led to substantial increases in the average peak load with both grout types though there was little change in the stiffness. There was also much closer alignment in the peak load between the resin and cement grout as shown in Table 3. The highest peak loads were achieved with the resin and cement grouted cable bolt in the standard borehole diameter and strong confining medium with average peak load of 397 and 406 kN respectively. The residual strength in this scenario was quite variable as shown in Figure 7 and much lower than in the weak confining medium at 99 kN. With the cement grout there was again close alignment between each of the test replications. The average peak and residual loads were much higher in this instance at 406 kN and at 216 kN respectively.

Figure 6: Load transfer behaviour of a cable bolt with resin grout (left) and cement grout (right) with an oversized borehole diameter in a weak confining medium.

Figure 7: Load transfer behaviour of a cable bolt with resin grout (left) and cement grout (right) with a standard borehole diameter in a strong confining medium.
Table 3: Comparison of test results between resin and cement grout in strong confining medium.

<table>
<thead>
<tr>
<th>Borehole diameter</th>
<th>Grout type</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>Residual strength at 90 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard (42 mm)</td>
<td>Resin</td>
<td>397</td>
<td>10.5</td>
<td>99</td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>406</td>
<td>10.3</td>
<td>216</td>
</tr>
<tr>
<td>Oversized (52 mm)</td>
<td>Resin</td>
<td>374</td>
<td>9.9</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>382</td>
<td>9.8</td>
<td>170</td>
</tr>
</tbody>
</table>

Oversize borehole diameter

The oversized borehole tended to have a more beneficial impact on the peak load anchorage of the cement grout than the resin grout and there tended to be less variability in the results for the resin and cement grout as shown in Figure 8. Despite this and unlike the performance observed in the weak confining material, the peak loads with both grout types was not as great in the oversized borehole. The average peak and residual loads reached 374 kN and 96 kN respectively, the latter being the lowest average residual load with use of the resin in the test program.

![Figure 8: Load transfer behaviour of a cable bolt with resin grout (left) and cement grout (right) with an oversized borehole diameter in a strong confining medium.](image)

The cement grout in an oversize borehole achieved an average peak load of 382 kN. The residual load as well as stiffness was of a similar level as measured in the other instances. While there was larger variation in the peak load with the cement grout, the variation in residual load was greatest with the resin grout.

**EFFECT OF BOREHOLE DIAMETER**

On the whole, borehole diameter had little effect on anchorage performance in the strong confining medium with peak load being on average 6% higher in the standard borehole diameter for both the resin and cement grout as shown in Table 4. Conversely in the weak confining material, the increase in borehole diameter resulted in substantial increases in peak load, residual load and stiffness. In the case of peak load, there were differences of 73% and 27% due to the increase in diameter for the cement and resin grouts respectively.
Table 4: Comparison of test results between resin and cement grout with different borehole diameters in weak and strong confining media.

<table>
<thead>
<tr>
<th>Confining medium</th>
<th>Grout type</th>
<th>Borehole diameter</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>% peak load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak 306 (15.7 MPa) Resin Standard</td>
<td>9.2</td>
<td>224</td>
<td>80%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oversized 369</td>
<td>16.3</td>
<td>328</td>
<td>84%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 27%</td>
<td>77%</td>
<td>46%</td>
<td>5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement 207 Standard</td>
<td>8.3</td>
<td>121</td>
<td>57%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oversized 358</td>
<td>9.9</td>
<td>138</td>
<td>38%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 73%</td>
<td>19%</td>
<td>14%</td>
<td>50%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong 397 (49.2 MPa) Resin Standard</td>
<td>10.5</td>
<td>99</td>
<td>25%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oversized 374</td>
<td>9.9</td>
<td>96</td>
<td>26%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 6%</td>
<td>6%</td>
<td>3%</td>
<td>4%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement 406 Standard</td>
<td>10.3</td>
<td>216</td>
<td>53%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oversized 382</td>
<td>9.8</td>
<td>170</td>
<td>44%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 6%</td>
<td>5%</td>
<td>27%</td>
<td>20%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

EFFECT OF STRENGTH OF THE CONFINING MEDIUM

Strength of the confining medium had more of an effect in the standard borehole diameter with increases of 30% and 96% in peak load for the resin and cement grouts respectively as shown in Table 5. The change was less significant in general in the oversized borehole.

Table 5: Comparison of results between weak and strong confining media with a resin and cement-based grout.

<table>
<thead>
<tr>
<th>Borehole diameter</th>
<th>Grout type</th>
<th>Confining medium</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>% peak load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard 306 (42 mm) Resin Weak</td>
<td>9.2</td>
<td>224</td>
<td>80%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong 397</td>
<td>10.5</td>
<td>99</td>
<td>25%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 30%</td>
<td>14%</td>
<td>126%</td>
<td>220%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement 207 Weak</td>
<td>8.3</td>
<td>121</td>
<td>57%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong 406</td>
<td>10.3</td>
<td>216</td>
<td>53%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 56%</td>
<td>24%</td>
<td>79%</td>
<td>8%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oversized 364 (52 mm) Resin Weak</td>
<td>15.2</td>
<td>257</td>
<td>70%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong 374</td>
<td>9.9</td>
<td>96</td>
<td>26%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 5%</td>
<td>54%</td>
<td>168%</td>
<td>169%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement 358 Weak</td>
<td>9.9</td>
<td>138</td>
<td>38%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong 382</td>
<td>9.8</td>
<td>170</td>
<td>44%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>change 7%</td>
<td>1%</td>
<td>23%</td>
<td>16%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

MEASURE OF STIFFNESS

Athanassiou (2016) and Hagan and Li (2017) reported values for stiffness in the order of 60 - 80 kN/mm. Their results are significantly higher than the stiffness reported in this study that is in the order of 10.5 kN/mm. This difference can be attributed to changes made in the procedure used in this test program. In both earlier cases, the test procedure provided for a constant embedment length in the pull-out test with 90 mm of shrink wrapping applied to the far end of the cable bolt before grouting. Further their tests involved grouting the entire free end of the cable bolt lying outside the confining medium within a thick-walled steel tube as opposed to fastening a bail and anchor at the end of the cable bolt as used in this program.
The former arrangement not only minimises the chance of any slippage during loading but also minimises the free length of cable bolt over which load is applied by the hydraulic actuator thereby the measured stiffness is of the grouted cable bolt section only.

CONCLUSIONS

The cement-based grout and resin grout were found to be equally effective in load transfer achieving the largest measured peak loads when placed in the high strength confining medium as shown in Figure 11. In the standard diameter borehole of 42 mm, the three-fold increase in strength of the confining medium from 15.7 MPa to 49.2 MPa resulted in 30% and 96% increases in average peak load with the resin and cement grouts to 397 and 406 kN respectively as shown in Table 6. Little differences were observed in the stiffness between all the test scenarios.

The resin grout tended to produce more consistent results with changes in borehole diameter and strength of confining medium. The cement grout was found to achieve a much lower average peak load in the weak confining medium and standard borehole diameter of 207 kN though there was a higher degree of variability observed between the test results in this condition.

![Figure 11: Summary of test findings indicating the highest load capacity was observed in the strong confining medium, the type of grout having little impact on the magnitude of the peak load. A larger diameter borehole had most effect in the weak confining medium only resulting in an increase in average peak load.](image)

**Table 6: Comparison of the effect of various test parameters sorted by peak load.**

<table>
<thead>
<tr>
<th>Residual strength at 90 mm</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>% peak load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strong Cement Standard</td>
<td>406</td>
<td>10.3</td>
<td>216</td>
</tr>
<tr>
<td>Strong Resin Standard</td>
<td>397</td>
<td>10.5</td>
<td>99</td>
</tr>
<tr>
<td>Strong Cement Oversized</td>
<td>382</td>
<td>9.8</td>
<td>170</td>
</tr>
<tr>
<td>Strong Resin Oversized</td>
<td>374</td>
<td>9.9</td>
<td>96</td>
</tr>
<tr>
<td>Weak Resin Oversized</td>
<td>364</td>
<td>15.2</td>
<td>257</td>
</tr>
<tr>
<td>Weak Cement Oversized</td>
<td>358</td>
<td>9.9</td>
<td>138</td>
</tr>
<tr>
<td>Weak Resin Standard</td>
<td>306</td>
<td>9.2</td>
<td>224</td>
</tr>
<tr>
<td>Weak Cement Standard</td>
<td>207</td>
<td>8.3</td>
<td>121</td>
</tr>
<tr>
<td><strong>mean</strong></td>
<td><strong>349</strong></td>
<td><strong>10.4</strong></td>
<td><strong>165</strong></td>
</tr>
</tbody>
</table>

Note: Strong and weak refer to strength of the confining medium. Standard and oversized refer to borehole diameter.
A 10 mm change in borehole diameter had less of an effect in the strong confining medium whereas in the weak material, an increase in borehole diameter increased average peak and residual loads. In practise, it is recommended that when installing a cable bolt in weak ground such as coal that the borehole diameter be increased slightly as this is more likely to improve the performance of the cable bolt to levels comparable to that observed in stronger rock such as sandstones and shale. The impact of the change in diameter in weak ground was found to be more acute with the use of cement grout than the resin grout.

ACKNOWLEDGMENTS

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MECHANICAL BEHAVIOURS OF GROUT FOR STRATA REINFORCEMENT

Ali Mirzaghornanali\textsuperscript{1}, Peter Gregor\textsuperscript{2}, Hamed Alkandari, Naj Aziz and Kevin McDougall

ABSTRACT: Past studies on mechanical properties of grout were critically investigated and classified. Small scale and large scale samples were cast using cube and cylindrical moulds. Samples were left undisturbed to cure for various time intervals ranging from 1 to 21 days. Effects of sample scaling on the Uniaxial Compressive Strength (UCS) of Minova Stratabinder HS were studied, using a universal compression testing machine. In addition, rectangular samples were cast to investigate bending resistance of the grout product. Four point bending test was carried out on the samples with curing time ranging from 1 to 21 days. It was found that compression resistance of the grout increased with respect to curing time and initial studies on flexural strength showed that the bending resistance of grout reduced with prolonged curing times.

INTRODUCTION

Rock bolt systems were first introduced for use as mining ground supports during the late 1940’s (Mark, 2017) in the form of mechanical anchoring. 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, unlike ordinary rock bolts (Mirzaghorbanali, et al., 2016). However, due to the increased use of non-metallic rock-bolts for rib support, cementitious grout has become a popular method for the anchoring of ordinary rock bolts. Due to similar grout preparation and installation processes, rock bolts are prone to similar premature failure of grout 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.

Previous studies in literature have focused on both the mechanical properties of grouts and resins (Aziz, et al., 2014; Mirzaghorbanali, et al., 2016) in addition to their encapsulation properties (Aziz, et al., 2016). Moreover, the studies have resulted in the determination of the mechanical 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 (\textit{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),

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- Modulus of Elasticity,
- Shear Strength and
- Creep/rheological properties.

Additionally, Hagan, et al., (2015) through the Australian Coal Association Research Scheme (ACARP) organisation investigated the effects of water to grout ratio on the UCS of both cylindrical and cube samples. The 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.

Furthermore, the study conducted by Mirzaghorbanali, et al., (2016) investigated the effects of curing time on the mechanical properties of grouts Jennmar BU 100 and Orica Stratabinder. Cube samples were prepared at tested at 1, 7, 14 and 28 days curing. The study concluded that:
- The compressive strength of grout increasing with curing time, and
- Both products are suitable for use in strata reinforcement.

**SAMPLE PREPARATION AND EXPERIMENTAL PROCEDURE**

The grout product Orica Stratabinder HS was selected to prepare the samples. The uniaxial compressive strength (UCS) samples were cast using two moulds; the small-scale 70mm cube mould and the large-scale 100 mm x 200 mm cylindrical mould. The flexural samples were cast using 350 mm x 100 mm x 50 mm prismatic moulds. Shown in Figure 1 is the mixing process for grout and the casting moulds. Samples were cast using a mixing ratio of 7 litres of water/bag and the application of slight vibration to remove trapped air. All samples were prepared at curing times of 1, 7, 14 and 21 days.

**EXPERIMENTAL RESULTS AND DISCUSSION**

**Uniaxial Compressive Strength (UCS)**

To conduct the UCS tests samples were prepared at 1, 7, 14 and 21 days curing times for each small and large-scale test as shown in Figure 2. Some tests were repeated to ensure accuracy of the collected data and the best four tests per sample type were presented.

The UCS values for large scale and small-scale samples at various curing times are presented in Figure 3. It is observed that the UCS values of the small-scale samples are higher than those of the large-scale ones for various curing times. The difference between the UCS of the large scale and small-scale samples is more pronounced for 7 days curing times where the UCS of the small scale is 68 MPa whereas the large-scale is 28 MPa. Figure 3 also indicates a delay in the strengthening process of the large-scale samples where the observed UCS difference between 1 day and 7 days for the large-scale test is just 2.1 MPa as opposed to 23.9 MPa for the small-scale test. The observed failure mechanisms were typical to that of those grout samples and presented in two stages. Failure can be identified by the formation of micro-cracks, which then leads to the second stage involving crack propagation.

The obtained UCS values show an increase in strength over the 21 day curing period for both small and large-scale samples, 44.1 MPa to 84.1 MPa and 25.9 MPa to 71.7 MPa respectively as shown in Figure 3.

The results of the two sample sizes were compared to determine a scale ratio. The Scale ratio is defined as the UCS value of small-scale samples to large-scale samples, and as shown in Figure 4 it varies from 1.1 to 2.4 depending on curing time.
Figure 1: [left] Grout preparation [right] a view of large scale UCS and flexural moulds

Figure 2: Prepared Samples [left] large scale cylinder UCS [right] small scale cube UCS

Figure 3: UCS values at 1 to 21 curing days [left] small scale samples [right] large scale samples
Three four-point bending tests were carried out on prepared samples at 1, 7 and 14 days curing time. Some tests were repeated to ensure accuracy of the collected data. Figure 5 [left] shows the 1, 7 and 14 days prepared samples using Stratabinder grout and [middle] and [right] present the testing process and sample failure respectively.

Shown in Figure 5 are the four point bending failure loads at various curing times. Unlike the UCS tests, the bending strength of grout decreased over time with initial values of 1.9 kN reducing to 0.5 kN throughout the curing period. The failure mechanism presented in the bending tests was identical to that of the UCS tests.
CONCLUSIONS

The experimental study found that the UCS of both small and large-scale grout samples increased with respect to curing time. For the seven-day curing times however, small-scale samples presented with better performance with significant strength improvement to that of the previous test when compared to the large-scale tests. Experiments indicated an increased overall response to curing times in the small-scale samples to that of the large-scale. It is noted that the result of large scale testing is more preferred for the sake of design in comparison to that of small scale testing. However, the small scale testing is the preferred method of testing due to its simplicity. Therefore the determined scale ration (1.7) allows for the results of small-scale tests to be easily converted to the large-scale scenarios for further Geotechnical implementation such as underground or slope stability analysis. Results of bending tests contradict the initial expectation based on which the peak-bending load should increase with an increase in the curing time. Therefore, it is suggested to carry out further investigation to study in detail the influence of curing time on bending properties of grout samples.

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NUMERICAL SIMULATION OF UNSATURATED INFILLED JOINTS IN SHEAR

Libin Gong\textsuperscript{1}, Jan Nemcik, Ting Ren

ABSTRACT: Rock discontinuities filled with soil-like materials commonly exist in rock masses, where the infill material is usually unsaturated, and the joint could have a much higher shear strength compared with fully saturated conditions. Understanding the shear behavior of unsaturated infilled joints is important when assessing ground stability such as in open cut mines or underground excavations. However so far, research on this topic is rare and not mature. This paper investigates the shear behaviour of unsaturated infilled joints in the numerical software FLAC. The FLAC soil-water retention and permeability models were modified in FISH subroutine to consider infill porosity change. A series of constant water content direct shear tests on infilled joints under various ratios of infill thickness to asperity height (\(t/a\)) were numerically conducted. Results highlight the necessity of correcting the intrinsic models in FLAC, and indicate that \(t/a\) ratio has a distinct influence on small-strain shear behaviour. Shear induced variations of fundamental infill parameters (e.g. matric suction, degree of saturation and saturated permeability) are discussed.

INTRODUCTION

Infilled rock joints are one of the most important geological structures in practice (Brady and Brown 2013). When infilled joints are located in arid regions and above the ground water table, the infill material could remain relatively dry and unsaturated (Barton et al. 1974). This may occur not only in shallow ground, but also deep underground, as in some places the ground water table can be located several hundred meters from the surface. Even below the water table, the infilled joints inside the rock masses may still be unsaturated. For example, an unsaturated zone may be generated around the underground roadways, due to the action of desaturation caused by increased permeability in the broken ground (Matray et al. 2007). However, the infilled joints are usually assumed as fully saturated in engineering practice for safety and convenience. Compared with saturated condition, the unsaturated infilled joints could have much higher shear strength. Hence it is of importance to understand the shear behaviour of rock joints filled with unsaturated materials. However so far, research on this topic is rare and not mature. In the shear process, some fundamental hydraulic and mechanical parameters such as infill degree of saturation, matric suction and permeability, would vary with shear displacement. Understanding the variation trends of these parameters is important for proposing the stress-strain constitutive models of unsaturated infilled joints. As far as can be determined nobody so far has investigated the variation trends during shear. Indraratna et al. (2014) conducted a series of triaxial shear tests on unsaturated infilled joints under constant water content conditions. An empirical shear strength model was developed considering the influence of infill degree of saturation. However the changes of infill saturation and suction during shear were neglected. Later Khosravi et al. (2016) studied the shear behaviour of unsaturated infill joints in the laboratory; however the infill matric suction was kept constant during shear, and thus the supposed variation of suction was not considered.

In fact it is very difficult to monitor the variations of those parameters such as degree of saturation, matric suction, and permeability during shear in laboratory tests. The environment at the shear area can be extremely harsh. Even though some sensors can be installed inside the joint infill for example the high-capacity tensiometers to measure matric suction, satisfactory contact between the sensor and the infill material and possible damage of the sensors still provide some challenging problems. This paper tried to investigate the variation trends of those fundamental parameters during shear in a numerical way. The numerical software FLAC/Two-Phase flow model was adopted to simulate the direct shear tests of unsaturated infilled joints (Itasca Consulting Group 2011). The influence of \(t/a\) (infill thickness divided by joint asperity height) on the shear behaviour was investigated. Some practical implications based on the obtained results are discussed.

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MODELLING PROCEDURES

This study modelled the 2D direct shear tests of unsaturated infilled joints in a laboratory scale using the software FLAC/Two-Phase flow model, under constant water content and constant normal load conditions. Simulated upper and lower rock parts of the joints were 20 mm high and 100 mm wide, respectively. Joints with a JRC value of 8-10 were modeled according to the Barton’s standard joint profiles. The infill thickness varies from 1.74 mm to 7.35 mm, corresponding to specified t/a ratios ranging from 0.5 to 2.5. A total of five tests were conducted at different levels of t/a, a normal stress of 0.5 MPa, and initial infill degree of saturation of 50%. A grid of 153 × 30 zones was built in the model. Model geometries with different t/a ratios are shown in Figure 1.

![Figure 1: Grid and interface plots of initial models under different values of t/a](image)

![Figure 2: Boundary conditions of the model](image)
Boundary conditions are plotted in Figure 2. Both sides of the upper block and the left side of the lower block were fixed in the x-direction; the bottom boundary was fixed in the y-direction. The boundaries of the infill material were impermeable to water. Pore air pressure was fixed as atmospheric in the system. After initial equilibrium, a horizontal velocity of \(1 \times 10^{-8} \text{ m/step}\) was applied to the lower block to produce a shear displacement.

In this paper, only the small-displacement shear behaviour was investigated, due to the limitation of FLAC. Total shear displacement was 1.5 mm to prevent contact between joint asperities, and the upper and lower rock parts were isotropic elastic material. The infill layer was modelled as a Mohr-Coulomb material. Deformability and strength properties required in FLAC for both the rock part and the infill part are listed in Table 1. Permeability, water retention parameters and fluid properties are listed in Table 2. The rock-infill contacting planes were described using the unglued interface model in FLAC, where the Coulomb shear-strength criterion was applied to detect shear failure. Adopted interface properties are also listed in Table 1.

### Table 1: Infilled joint specimen properties required in FLAC

<table>
<thead>
<tr>
<th>Properties</th>
<th>Rock parts</th>
<th>Infill layer</th>
<th>Interfaces</th>
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<tr>
<td>Constitutive model</td>
<td>isotropic elastic</td>
<td>M-C model</td>
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<tr>
<td>Dry density (kg/m(^3))</td>
<td>2500</td>
<td>1500</td>
<td></td>
</tr>
<tr>
<td>Elastic drained bulk modulus, (K) (Pa)</td>
<td>10.65(\times 10^6)</td>
<td>7.8(\times 10^6)</td>
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<tr>
<td>Elastic shear modulus, (G) (Pa)</td>
<td>4.36(\times 10^9)</td>
<td>5.8(\times 10^8)</td>
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<tr>
<td>Poisson’s ratio</td>
<td>0.32</td>
<td>0.2</td>
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</tr>
<tr>
<td>Drained cohesion, (c) (Pa)</td>
<td>-</td>
<td>10(\times 10^3)</td>
<td>10(\times 10^3)</td>
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<tr>
<td>Drained friction angle, (\varphi) (°)</td>
<td>-</td>
<td>17</td>
<td>20</td>
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<tr>
<td>Dilation angle</td>
<td>-</td>
<td>0</td>
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<tr>
<td>Tension limit (Pa)</td>
<td>-</td>
<td>0</td>
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<tr>
<td>Initial porosity, (n_0)</td>
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<td>0.544</td>
<td></td>
</tr>
<tr>
<td>Shear stiffness, (k_s) (Pa/m)</td>
<td></td>
<td>3.1(\times 10^{10})</td>
<td></td>
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<tr>
<td>Normal stiffness, (k_n) (Pa/m)</td>
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<td>3.1(\times 10^{10})</td>
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### Table 2: Permeability, water retention parameters and fluid properties

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<th>Properties</th>
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<td>Van Genuchten parameter, (a)</td>
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<td>Van Genuchten parameter, (c)</td>
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<tr>
<td>Gallipoli parameter, (\psi)</td>
<td>4.117</td>
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<td>Fluid modulus for water, (K_w) (Pa)</td>
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<td>Fluid modulus for air, (K_g) (Pa)</td>
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<td>Residual degree of water saturation, (S_{res})</td>
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<td>Undrained coefficient, (\beta)</td>
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<td>Viscosity ratio, (\mu_w/\mu_g)</td>
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### POROSITY CONSIDERATION

In unsaturated soil mechanics, it is evident that porosity has significant influences on the Soil-Water Retention Curves (SWRCs) and permeability (Mašín 2010; Gallipoli, et al., 2015; Hashem and Houston 2016; Carrier and Beckman 1984). However in FLAC/ Two-Phase flow model, these porosity dependencies are not accounted for automatically. To obtain more accurate simulation results, the intrinsic constitutive models were modified through FISH subroutines.
Porosity calculation

As FLAC does not calculate the volume change induced porosity variation, the real porosity was first obtained in FISH functions.

In the Mohr-Coulomb model, elastic volumetric strain \( e^e \) is

\[
e^e = \frac{\sigma_1 + \sigma_3}{\sigma_1 + \sigma_2}
\]

where \( \sigma_1 \) and \( \sigma_3 \) are the major and minor principal stresses, respectively; \( \alpha_1 = K + 4G/3 \) and \( \alpha_2 = K - 2G/3 \), where \( K \) is drained bulk modulus and \( G \) is shear modulus.

The volumetric strain in the large strain mode can then be approximated by

\[
e^e_{\text{large}} = \frac{2e^e}{2+e^e}
\]

The principal stresses in Eq. (1) are calculated by converting the Cartesian stress components available within FISH:

\[
\sigma_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \frac{1}{2} \sqrt{(\sigma_{xx} - \sigma_{yy})^2 + 4\sigma_{xy}^2}
\]

\[
\sigma_3 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \frac{1}{2} \sqrt{(\sigma_{xx} - \sigma_{yy})^2 + 4\sigma_{xy}^2}
\]

where \( \sigma_{xx}, \sigma_{yy} \) and \( \sigma_{xy} \) are three components of stress tensor in the Cartesian coordinate frame.

The FISH variable \( e_{\text{plastic}} \) represents the accumulated plastic volumetric strain relating to the shear yield surface. Hence the total volumetric strain \( e_v \) is

\[
e_v = e^e_{\text{large}} + e_{\text{plastic}}
\]

Porosity \( n \) can then be approximated as

\[
n = 1 - \frac{2-e_v}{2+e_v}(1-n_0)
\]

where \( n_0 \) is the initial porosity of the material.

Porosity-corrected SWRC

FLAC adopts the conventional van Genuchten SWRC model:

\[
s = P_0\left[S_r^{-1/\alpha} - 1\right]^{1-\alpha}
\]

where \( P_0 \) is a parameter relating to surface tension, intrinsic permeability and porosity of the material, \( S_e \) is the effective degree of water saturation, and \( s \) is matric suction of the porous material.

To consider porosity in Eq. (7), the residual degree of saturation and \( P_0 \) are defined as:

\[
S_{res} = 0
\]

\[
P_0 = \frac{(1-n)^\psi}{\varphi \cdot n \cdot \psi}
\]

where \( \varphi, \psi \) are constant parameters.

Equation (7) is then transformed to the Gallipoli SWRC model (Gallipoli et al. 2003):

\[
s = \frac{(1-n)^\psi}{\varphi \cdot n \cdot \psi} \left[S_r^{-1/\alpha} - 1\right]^{1-\alpha}
\]
Porosity-corrected permeability

Carrier and Beckman (1984) showed that porosity or void ratio of remolded clay has an influence on its saturated permeability:

$$k_s \approx 0.0174 (1 - n) \left\{ n^{0.027 (PL - 0.242 PI)(1 - n)} \right\}^{4.29}$$ (11)

where $PI$ is plastic index for the material, and $PL$ is plastic limit for the material.

As the laboratory infilled joint specimens were simulated in this study, and the infill material is considered as remolded clay, Eq. (11) was used in FISH subroutine to correlate saturated permeability with porosity.

Unsaturated flow - mechanical coupling

In Two-Phase flow model, the unsaturated flow and mechanical calculation is coupled as shown in Figure 3.

RESULTS AND DISCUSSION

Porosity correction

As described above, porosity was considered in the SWRC model, saturated permeability and mechanical stiffness calculations. However it is too time-consuming to conduct the porosity correction at every calculation step. Hence this section investigates the influence of the number of porosity-update times, or the correction frequency during the whole shear process, on the infilled-joint shear behaviour. This number varies from zero (uncorrected for displacement of 1.5 mm), to 15 times (once per 0.1 mm shear displacement), to 30 times (once per 0.05 mm), and to 60 times (once per 0.025 mm).
All graphs in Figure 4(a-f) clearly show that the porosity correction has a significant influence on the shear behaviour. After correction, a strain softening behaviour is clearly observed, matric suction decreases and the saturated permeability increases significantly, compared with the original case. Once corrected, the updating frequency has little effect on the shear stress curves. With increase of the frequency, dilation decreases, degree of saturation becomes larger, matric suction drops, and saturated permeability increases. Peak shear strength decreases with the increase of update frequency, and remains stable when the number is beyond 30. Therefore this value is adopted for the following simulation.

\( t/a \)

Tests under various \( t/a \) values (0.5, 1.0, 1.5, 2.0, 2.5) were then simulated and results are shown in Figure 4 (g-l). It is obvious that with the increase of \( t/a \) ratio, the strain softening behaviour diminishes, the infill is compressed more, the changes of degree of saturation, matric suction and saturated permeability all became gentler and peak shear strength decreases in an exponential form.

**PRACTICAL IMPLICATIONS**

As discussed, study of the influence of porosity correction represents the inaccuracy of the FLAC intrinsic models. The residual shear strength may be overestimated considerably if the original FLAC models are used. This could cause an unsafe stability analysis for the jointed rock excavations in practice.

As this study focuses on the shear behaviour of infilled joints before joint asperities contact, Fig. 4(l) suggests that even the “soil peak” in the whole shear stress-shear displacement curve is sensitive to the \( t/a \) ratio. This is because with the increase of \( t/a \) ratio, the obstruction from the rough joint profile against the infill squeezing (flow) becomes weaker, and the maximum stress concentration factor within the infill layer decreases. This is important particularly in some circumstances where the infill layer is relatively large. Understanding the influences of \( t/a \) ratio on the “first stage” of the infilled joint shear may benefit the stability evaluation and reinforcement design.
Figure 4: Shear behaviour of infilled joints under different porosity-update frequencies and t/a values
CONCLUSIONS

This paper employed the FLAC Two-Phase flow model to conduct a series of direct shear tests of unsaturated infilled joints under Constant Normal Load (CNL) and Constant Water content (CW) conditions. The following conclusions can be drawn:

1. Porosity correction in FLAC Two-Phase flow model is essential as the original constitutive models could overestimate the residual shear strength.
2. Generally during shear: stress increases to a peak and then decreases or remains stable; the infill layer contracts after an initial small dilation, followed by dilation or continued compression depending on the t/a ratio; degree of saturation increases and then remain steady or decreases; matric suction reduces with decaying slope especially after peak shear strength; saturated permeability increases significantly in the later stage exponentially.
3. With the increase of t/a: peak shear strength decreases exponentially; both the strain softening and dilation behaviour is less obvious; and the changes in degree of saturation, matric suction and saturated permeability all becomes smaller.

ACKNOWLEDGEMENTS

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<td>Zhu, Weibing</td>
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