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FOREWORD

The organizing committee of the Coal 2008 is pleased to hold, once again, the 8th Coal Operators Conference at Wollongong. This year the conference is held in February, a favoured month to hold the conference. We are also pleased to have participants from China, India and Iran; this is a healthy sign of the increasing awareness of the Coal Operators Conference standing in Australia and beyond.

A total of 31 papers in the conference proceedings are from different field of mining operations, ranging from longwall mining, ground control, mine gases and outburst control, spontaneous combustion, mine dust control and others. The interest in the importance of the conference is reflected on the quality of papers presented and this conference is now being established as a popular venue for reporting on new technologies introduced to the industry for the betterment of mine production, productivity and safety. It is pleasing to note that the Australian coal mining production for the past 12 months was up from 300 mt – 317 mt and fatality free, a record which places Australia in the top league for mine safety.

This year Xstrata Coal, BHP Billiton, Ellton Group, Groutech, Gujarat NRE Minerals Limited, Jennmar Australia and Minova Australia and are the sponsors of the conference. This is a welcome sign which is a reflection on the growing status of the conference series, as the main forum for the exchange of ideas between mine operators, engineers, consultants and researchers in the diverse field of coal mining technology.

We would like to express our sincere thanks to:
- The organizing committee members for their diligence and hard work in making this conference a success,
- The authors of the papers, who have taken considerable time and effort in the preparation of their papers to the required standards
- The reviewers of the papers, which at times has not been an easy task, but ensured the high standards of the papers being maintained,
- Peter Vrahass and his colleagues at the Uni-Centre of the University of Wollongong for the management and registration of the conference. Peter is to be congratulated for setting up the Coal 2008 Conference web site,
- Leonie McIntyre of the Faculty of Engineering, University of Wollongong for type setting the conference proceedings,
- Barry Robertson for audio–visual management of the conference venue, and
- Staff of The Wollongong University Printery for printing the conference proceedings and to Gerard Toomey for designing the proceedings covers.

Naj Aziz (Conference Convenor and Editor)
Jan Nemcik, (Co-editor)
PREFACE

Coal 2008 represents an ideal forum for industry people to gather, share experiences and understand recent advances in a wide range of coal mining related disciplines. By holding this conference in Wollongong, it is our desire to maintain an operational focus, provide a pleasant environment and wherever possible solicit the direct involvement of those who are actively “doing the work”.

The range of papers presented reflect the diversity of issues or “challenges” that confront all underground coal mining operations, especially those in the Illawarra. The fundamental need to improve productivity is a strong incentive that has driven all Illawarra mines and reflects upon their ability to utilise and adopt new technologies, and new techniques to remain operational and where possible stay ahead of the field. However the most important and significant change impacting upon all coal mines has been the focus on safety. This has necessitated a change in thinking on how work is undertaken, the risks relating to any and every aspect of the mining operation, the controls needed to ensure no worker is injured and the provision of appropriate training for all personnel on how a particular job is to be undertaken. There now exists an expectation that people should not and will not be injured.

The Wollongong University and Coal 2008 offer an ideal venue for mine operators, equipment suppliers and research scientists to talk, compare notes, findings, case studies and improve their knowledge base. In promoting this opportunity to share ideas, special thanks must go to the authors and presenters of papers, the organising committee and to our sponsors, Xstrata Coal, BHP Billiton, Eliton Group, Groutech, Gujarat NRE Minerals Limited, Jennmar Australia and Minova Australia. Without the support of these people and the generosity of these companies, the success of the conference would not be guaranteed.

As the Chairman of the Illawarra Branch for the Australasian Institute of Mining and Metallurgy it gives me great pleasure to welcome all the delegates to “Coal 2008”. I trust that your time spent at the conference is enjoyable, productive and that in some small way our mines are safer and more productive as a consequence.

Dr Chris Harvey
Chairman,
Illawarra Branch, Aus IMM.
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MANAGING ROOF CONTROL PROBLEMS ON A LONGWALL FACE

Robert Trueman\textsuperscript{1}, Geoff Lyman\textsuperscript{1} and Alan Cocker\textsuperscript{1}

ABSTRACT: A proven way of interpreting the shield leg pressure sensor data within each shield load cycle has been developed by the authors and this has been encapsulated into real time and non real time software. A load cycle is the change in support pressure with time from setting the support against the roof to the next release and movement of the support. It is now possible to automatically identify when a shield has too low a set pressure, and when a shield is faulty and/or has an inadequate capacity for the conditions. It has been found that once set conditions deteriorate and shields are set manually it is very common for set pressures to be too low for the conditions, resulting in roof control problems. The software can automatically identify set pressures that are too low which will enable auditing of shield operation and corrections to be made. Up to 10\% of shield legs have faults on a typical Australian longwall and these periodically result in localised roof control problems. Faulty support components are automatically identified, enabling timely repairs to be made. On some longwalls the shields become overloaded at the peak of the periodic weighting cycle and the software can identify the difference between a heavily loaded support and one that is overloaded. By minimising the cycle time and making sure that set pressures are adequate in cycles following the overloading event, it is quite possible to successfully mine through an overloading event if the event is correctly identified.

The use of this software has the potential to significantly reduce or even eliminate roof control problems on a longwall face with significant benefits to both productivity and safety. By automatically identifying potential causes of roof control problems and offering solutions, the software has the potential to aid longwall automation. A Beta test version of the real time software has been successfully working at BMA’s Broadmeadow Mine for some time and several mines have benefited from expert off-line analyses using the software. The software can also be used to isolate the many interconnected factors affecting roof control on a longwall face, which will enable their quantification and is therefore a powerful research tool.

INTRODUCTION

In recent years, longwall roof supports (shields) have been equipped with pressure sensors and hydraulic leg pressures can be displayed in real time. Despite the vast quantity of monitoring data captured on a daily basis from the majority of modern longwall faces, few geotechnical analyses are undertaken. The collected data are largely unused for roof control or for other purposes such as maintenance. The failure of the industry to make full use of the data from the face has largely been a result of its volume in real time, its level of corruption, and the lack of a methodology for data interpretation in the context of support-strata interaction.

It is now possible to go to a longwall site and, using the software developed by the authors, interpret how the supports are interacting with the strata without any prior monitoring of the longwall and to identify faulty support components. This allows the optimisation of mining strategy in terms of set pressure and maintenance scheduling in order to mitigate or even avoid strata control problems. Issues such as faulty support components have been demonstrated to periodically lead to roof control problems and costly delays at all sites, even those where roof conditions were good. Setting the supports too low for the conditions can also lead to significant roof control problems.

The interpretations and identifications are based on the recognition of characteristic load cycles. A load cycle is the change in support pressure with time from setting the support against the roof to the next release and movement of the support. The recognition of the load cycles depends on accurate delineation of roof support pressure behaviour and extraction of key features such as average pressure throughout the cycle, the number of yield events, the set pressure, the rate of loading in part or all of the cycle and the cycle length. The extensive validation of the capabilities of the software showed that load cycle features were accurately mapped and that they could give a useful interpretation of the mining conditions.

In contrast to the software available prior to the authors’ developments, the system uses and cross references all available data to obtain the required accuracy of load cycle definition. The minimum data input requirement is shearer position, DA ram extension and pressure sensor data, but AFC and shearer power draw can also be used to improve the accuracy of interpretation and aid in identification of corrupted signals. Existing software were all found to be incapable of accurate load cycle delineation, which significantly limits their interpretative capabilities. This is not to say that these software packages are not useful, rather that interpretive capabilities based on load cycle analysis cannot be encapsulated into those programs.

\textsuperscript{1} BRC Mining and Geology, University of Queensland
The software developed by the authors of this paper has now been extended from “off-line” to “on-line” as an aid to managing roof control problems as they develop.

LOAD CYCLE ANALYSIS

A number of geotechnical models have been developed that claim to be able to relate how a longwall powered support is interacting with the surrounding strata. The methods include: detached block theory (Wilson, 1975); yielding foundation theory (Smart and Aziz, 1986); empirical nomograph based method (Peng et al, 1987); load cycle analysis (Park et al, 1992; Peng, 1998); neural networks (Chen, 1998); various numerical models (eg Gale, 2001; Klenowski et al, 1992); ground response curves (Medhurst and Reed, 2005). All models in the public domain literature were reviewed by the authors of this paper (Trueman et al, 2005a). It was concluded that the above models, whilst offering important contributions towards understanding support-strata interactions, did not offer effective means of interpreting how supports interacted with the strata. Periodic weighting and time dependency are essential components of any model that attempts to describe shield-strata interaction and none of the above models considers both of these and most do not consider either. Support yielding is also important and is seldom explicitly considered.

A mechanistic consideration of how the support and strata interact indicates that there should be four basic pressure time patterns to be observed that will indicate when a support has: an adequate capacity and appropriate set pressure; adequate capacity and too high a set pressure; inadequate capacity; and too low a set pressure. The concept has been validated and further refined through the back-analysis of more than 700,000 individual load cycles on seven longwall panels located in six seams at five mines and nine geotechnical domains.

Adequate capacity and appropriate set pressure

A load cycle in which near stabilisation of the roof is achieved, without one or more yielding events (Figure 1), indicates that the roof characteristics are such that the set pressure is adequate to ‘clamp’ the roof in place or to support the roof load without any downward displacement other than that due to the elastic compression of the support. There is an initial rapid increase in support pressure followed by a marked decrease in the rate of pressure increase as the support stabilises the roof. The term ‘near stabilisation’ is used as full stabilisation (no further convergence) may never occur in a normal cycle time.

Minimal roof deterioration is associated with such pressure-time profiles. There should be minimal, if any, roof control delays and hence production rates may be high.

Time should not play an important role in terms of the support-strata interaction for a support with an adequate capacity and appropriate set pressure. If the load cycle is short then the portion of the pressure time cycle where near stabilisation is occurring will be shorter and will be longer for longer load cycles. Delays in production, for a maintenance shift for example, should not adversely impact on roof control.

Too low a set pressure

Where set pressures are too low for the conditions loading rates are high when compared to situations where the support capacity is adequate and the set pressure is appropriate. The loading rate remains relatively constant throughout the load cycle and the support is unlikely to reach yield. Figure 2 illustrates a typical pressure time chart where set pressure is too low for prevailing conditions; the pressure rise from set to release is relatively constant with little indication of a reduction in the rate of loading. Under such conditions increased loading rates caused by the set pressure being too low for the conditions may result in the unraveling of roof strata. This unraveling results in degradation of the roof conditions such that at the release of the support the roof would not be able to maintain its integrity and would start to break up, leading to difficult set conditions in the next cycle. The difficulty in setting
the supports may further exacerbate the problem, as set pressures achieved on the subsequent load cycles would be affected by the poor roof conditions such that achievement of sufficient set pressures may not be possible. The low set pressures and associated rapid roof movement and unraveling of the strata may eventually lead to roof cavities and potential production delays.

![Graph](image)

**Figure 2 - Typical loading pattern with set pressure too low for the conditions**

In the authors’ experience too low set pressures usually results where set conditions are poor and operators resort to manual setting of supports. This can be due to soft and fractured immediate roof due to say the presence of faulting or where the longwall has been stopped for a considerable time where the immediate roof is weak. Where supports have been overloaded, as described later, set conditions for subsequent cycles can become difficult.

Extensive back-analyses of longwall sites in Australia carried out with the aid of the software has revealed that once manual setting is resorted to it is quite common for operators to give little consideration to the set pressure. In the authors’ experience low sets can often be avoided and are often a result of operator’s lack of understanding of the effects that set pressures below a critical value can have on roof control. Even where set conditions are poor it is usually possible to set supports to an adequate set pressure but this can take more effort on the part of the operator. The premature overriding of automatic setting of supports also appears to be an endemic problem on Australian longwalls. Routine auditing of set pressures and when automatic setting is overridden is essential for managing roof control problems on a longwall face and the software described in this paper facilitates this.

In long load cycles the support pressure may reach a level which would normally be adequate for the conditions. However by this time damage to the roof will have already occurred. Minimising the cycle time where set pressures are too low for the conditions leads to less roof degradation. Wherever possible increasing set pressures is the best strategy for minimising roof control problems where set pressures are low. The premature overriding of automatic setting of supports also appears to be an endemic problem on Australian longwalls. Routine auditing of set pressures and when automatic setting is overridden is essential for managing roof control problems on a longwall face and the software described in this paper facilitates this.

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Once roof conditions deteriorate to such an extent that the roof begins to break up the pressure-time signals often become more complex than shown in Figure 2 when set pressures are low. Nevertheless, control of the roof is seldom if at all achieved until set pressures return to acceptable levels, even if remedial action is taken such as grouting the roof. The authors have observed many situations on longwalls where cavities have continued to form after remedial work has been carried out because set pressures have been low in subsequent cycles.

Roof control problems are not inevitable with set pressures below 40 tonnes per square meter because time is a critical factor as previously discussed. If mining is rapid it is quite possible that roof control problems may not develop. At some stage however delays to production will occur and if set pressures are not adequate roof control problems will eventuate.

**Inadequate support capacity**

When there is no evidence of stabilisation during support yielding, as in Figure 3, the support cannot arrest the roof movement and the support capacity is inadequate for the conditions. Two causes of support overloading have been observed by the authors. One is due to loading transferred from the immediate and/or main roof. The second is a result of one or both supports adjacent to a particular support being faulty and therefore not carrying their rated load. The functioning support is forced to take a higher load in such a case. In such circumstances average loading rates tend to be high.
Based on the experience of the authors, a typical Australian longwall where the supports are not new will have up to 10% of the shield legs faulty at any one time. This is resulting in significant localized roof control problems and production delays. The automatic identification of faults by the software is aimed at achieving rapid repair and a minimization of these problems.

The phenomenon of ‘periodic weighting’ occurs when the main and/or immediate roof cantilevers over the support. Where competent beds occur in the main and/or immediate roof long cantilevers can form over the support and, as the cantilever lengthens, loads are created which simply cannot be resisted by the support. In such conditions loads just after the cantilever breaks off are usually moderate and can be stabilised, but the load just before the cantilever breaks where periodic weighting intervals are long in physical extent may exceed the support capacity.

The authors have observed effects from thick competent beds whose base was located 50 m from the top of the extraction and the top of which extended to 80 m into the roof at one mine. At another mine the authors identified a thick competent bed as the cause of shield overloading that was located between 65 m and 85 m into the roof. Both mines were extracting a thickness of 4.5 m. This appears to mean that shield loading can be affected by beds located within 20 times the extraction height.

The load cycle characteristic typified by Figure 3 usually only occurs within the high loading portion of the periodic weighting, which normally over 1 to 3 shears. In the low loading portion of a periodic cycle, the cycles, ideally, are similar to those in Figure 1.

When the supports are overloaded, the yielding events lead to significant roof to floor convergence. This convergence increases the probability of roof guttering between the support tip and coal face, making the resetting of the support difficult for the next cycle, and leading to the break up of the immediate roof strata above the supports. Depending on the speed at which mining advances the associated roof degradation can lead to serious roof cavities, resulting in lengthy production delays. The difficulty in resetting the supports may lead to a failure to achieve adequate set pressures for the supports, further compounding the roof control problems. This issue has been discussed above. The combination of roof guttering due to multiple yield events resulting in poor set conditions and low set pressures on subsequent shears is what usually results in roof cavities. Cavity formation therefore generally evolves over two or three load cycles. However, if cycle times are long enough, say where mining has stopped for planned maintenance or an equipment breakdown, roof guttering can deteriorate into cavities within a single load cycle.

Time is a very important parameter in a loading pattern described by Figure 3, irrespective of the cause of the support overloading. If mining proceeds rapidly, the number of yield events and the extent of convergence will be minimised, potentially limiting the extent of guttering between the support tip and face. If mining is rapid enough, it is possible in many situations to get through the interval of support overloading without significant roof control problems. When supports are overloaded it is not appropriate to stop the face for scheduled maintenance, for example. This appears to be an obvious point but the authors have back-analysed problem areas at longwalls using the concepts outlined in this paper where this has happened – simply because operators were unaware that supports were overloaded. After an overloading event, every effort should be made to achieve reasonable set pressure on subsequent cycles under what may be difficult set conditions as described previously. Identifying faulty supports and their timely repair will minimise the possibility of localised roof control problems under all roof conditions.

**Figure 3 - Support loading for yielding without stabilisation indicating a support with inadequate capacity.**

There appears to be a belief in the Australian mining industry that the potential for roof control problems on a longwall face increases with the number of shields that are in yield. This is only partially correct. Through extensive back-analyses at a number of sites the authors of this paper have recognised that it is actually the number of yield cycles that occur in individual support load cycles that influences roof control. If there are less than three yield
events in an individual load cycle then roof degradation is seldom observed. In general fretting of the roof between
the shield tip and face is observed after about three yield events and as the number of yield events increase this
deteriorates into guttering and then cavities. This has been observed where only a small number of shields have
been forced into overload because nearby shields were faulty and not able to carry their rated load. On the other
hand no roof control problems were observed in a case study where two thirds of the shields on the face were in
yield because the number of yield events in the load cycle were less than three in that particular case. Obviously
more roof control problems can be expected where a large number of shields are experiencing a large number of
yield events.

Because time plays such a critical role in shield-strata interaction it is often not possible to directly equate roof
control problems to the causes; be they overlying strata, set pressure or maintenance related. If mining is rapid roof
control problems may not develop with faulty shields, where shield set pressures are inadequate or where there
are thick competent beds in the roof. But eventually cycle times will be long enough to cause roof control problems
where one or more of these factors exist.

**Very high set pressures**

Set pressures as high as 90% of yield is not uncommon in Australian mines when automatic set is engaged and
there appears to be a belief that this leads to improved roof control. The authors of this paper have never observed
any problems relating to set pressures when shields are set to at least 60% of yield for shields with a support
density of 100 tonnes per square meter or greater before the cut. In general the authors also have not seen any
roof control problems relating to high set pressures. In some situations shields have drifted into yield but in the
absence of thick competent beds in the immediate or main roof the shields usually have undergone only a single
yield event and no degradation to the roof was observed. However, there are situations where very high set
pressures may contribute to poorer roof conditions.

Where periodic weighting is high enough to result in periodic shield overload it may be better for set pressures to
be nearer 60% of yield than 90% (with shields of support density of 100 tonnes per square meter or above). This
relates to the effect of time. If the support is set to 60% of yield then it will take much longer to get to the first yield
event and for the same cycle time there will be fewer yields. Fewer yields will result in less convergence and
subsequent roof degradation and it will be easier to mine through the periods of support overload. If a shield
periodically has an inadequate capacity for the conditions the authors have seen no evidence that very high setting
loads will stabilise the roof. The belief that very high set pressures are beneficial may have arisen when support
capacities were less and set pressures close to the yield value were necessary for the set pressure to be
adequate.

**Panel width, extraction height and seam depth**

A number of authors have concluded the need for a greater powered support capacity with increasing panel width,
extraction height and depth (eg Medhurst and Reed, 2005; Frith and Creech, 1997). Nevertheless the impact of
these factors on support loading is still debated. For example Barczak (2006) challenged the need for increased
shield capacity in higher extraction height longwalls. The authors of this paper have also analysed a number of
relatively shallow longwalls where support overload was occurring even with relatively high capacity shields. Shield
loading is a complex interaction between: shield capacity and set pressure, the composition of the main and
immediate roof, the presence or absence of leaking legs, extraction height, cycle time, panel width and seam
depth. It has proven very difficult to isolate all these factors. The software developed by the authors that
encapsulate their load cycle analysis theories now allows a rapid identification of the causes of shield loading and
should facilitate a much better understanding and quantification of the different factors.

One of the factors affecting shield loading that is seldom mentioned is the effect of time. Where longwalls mine
wide panels and/or thick seams this inevitably means that individual load cycles are greater than for thinner seams
and narrower panels.

This will therefore lead to greater potential for roof control problems to develop at times when set pressures are
inadequate, in the vicinity of faulty supports and in situations where periodic shield overload occurs. Auditing set
pressures and timely repair of faults will therefore have a larger positive impact on roof control in wider panels and
higher extraction heights. That is not to say that auditing set pressures and identifying leaking legs are not
important for all extraction heights and panel widths.

Where shields are overloaded at or near the peak of the periodic weighting cycle, the longer cycle times associated
with an increase in panel width and/or extraction height may lead to more roof control problems. The merits of
increased panel width may of course offset the potential for increased roof control problems in such conditions.
Nevertheless, when assessing panel width it would appear wise to understand the potential for periodic shield
overload. Recognising such events will also increase in importance with an increase in panel width.
SHIELD CAPACITY

Over the last 20 years hydraulic support capacities have increased significantly. For example, in 1984 the average support capacity in the US was about 450 tonnes and the maximum was about 730 tonnes (Barczak, 2006). By 2005 the average was about 800 tonnes and the maximum was 1160 tonnes. The greater canopy areas to accommodate one web back and wider supports means that support densities have not increased as much however. Most current longwall mines in Australia had a support density in the region of 100 tonnes per square meter before the cut until recently. The newer supports tend to be rated at about 115 tonnes per square meter and Moranbah North have on order 1750 tonne capacity supports with a support density of 140 tonnes per square meter. The technology is there to significantly increase support capacities still further, the current limitation apparently being the ability of the OEMs to test the supports. If the demand for higher capacity supports is there then that limitation will be overcome, but at a price.

Whilst it is true that roof control on a longwall face has improved over the years it is debatable what role support capacity alone has played in this and if further increases in capacity will lead to further improvements. Using the new concepts of load cycle analysis encapsulated in software it is now possible to answer those questions. For the first time it is now possible to rapidly differentiate between causes of roof control problems that relate to poor operation of the support, maintenance problems or those relating to strata-support interaction.

SOFTWARE DEVELOPMENTS

The commercial software being used to manipulate the pressure data that existed prior to the developments described in this paper were found to be deficient in the type of load cycle analysis proposed herein, simply because the necessary concepts for load cycle analysis had not been developed at the time of their release. Analyses of this software (Trueman et al, 2005a) concluded that they were incapable of achieving accurate load cycle delineation. Neither had they any capability for extracting the critical load cycle features essential for interpreting how supports were interacting with the roof strata or differentiating between accurate and corrupt signals. This is not to say that the software packages are not useful, merely that the potentially powerful concepts relating to load cycle analysis noted above cannot be incorporated into these programs.

Existing software used only the pressure signals. The authors' new code uses the pressure signals, DA ram and shearer positions as a minimum. Extensive manual verification of events on the longwall identified by the code has shown that the cycle identification algorithms have a very low error rate and are able to correctly reject corrupt sensor data (Trueman et al 2005b). At present, the accuracy of the code is unmatched. Other signals such as AFC and shearer power draw can be used to further increase the accuracy of load cycle identification when they are available. The code is capable of negotiating signal drop-out, which has been found to be a problem on many longwall faces, and is able to accurately track the spatial position of the face in terms of metres of retreat.

Additional algorithms were developed to extract the critical load cycle features that are essential for interpreting support-strata interaction. The following features are extracted from the accurately determined load cycle for every support leg on the face:

- map of the time weighted average pressure (TWAP)
- map of yield events
- map of the set pressures lower than a user defined threshold
- map of cycle times (time from set to release)
- map of loading rate in parts of the load cycle
- identification of when posi-set has been activated
- map of support legs not carrying their full rated load due to faults associated with yield or check valves
- map of noisy pressure sensor signals that indicate incipient sensor failure.

Extracting such features accurately is not a simple matter given the noisiness of the signals and the degree of signal corruption or loss experienced on many longwalls. Figure 3 provides a good example of the degree of difficulty in isolating yield events from the pressure signals. Elements of the code that detect such features must be very carefully designed and tested to ensure that they identify the sought-after events while rejecting pressure variations that do not represent a yield. Identifying every load cycle on every shield is likewise not a simple task.

In the first instance an "off-line" version of the code was developed in order to validate the load cycle analysis concepts and demonstrate the interpretive capabilities of the software.

Validation of load cycle analysis concepts

To use the load cycle characteristics described above with confidence as an analysis tool, they had first to be validated. It had to be shown that the characteristic pressure-time profiles described above do indeed exist in real mining conditions and the anticipated roof control conditions associated with each do in fact occur. A comprehensive assessment of the load cycle analysis concept at a range of specifically selected longwall mines
allowed for such a verification process. Validation revealed excellent correlation between the roof control conditions predicted to be associated with each cycle type and the actual roof control conditions.

The research relied upon analysis of support loading cycles for every support across the face over a total advance at all the mines in excess of 2 km. Approximately 700,000 loading cycles were analysed. This analysis was made possible by development of the new software described above.

**Analysis of spatial roof support loading patterns “off-line”**

In addition to permitting the validation of the load cycle concepts developed by the authors, the new software has been used to assess support strata interaction, allowing rapid adaptation of mining strategies and set pressures for optimal roof control and maintenance scheduling. The current capabilities of the software are illustrated from an example. Data that was made available on support loading at Mine A between the 10th July and 7th August 2005 was run through the software. An interpretation of the support-strata interaction for this period was made using the outputs generated by the software.

The data are presented as maps in which the x-axis is the leg number counting from the left (tailgate end) of the longwall face which is on the left hand side of the diagrams. The y-axis is the shear number in the direction of mining and is therefore proportional to mining advance.

The plotted value is the value of the variable of interest and it is usually coded by colour. Figure 4 is the TWAP (bar) for the period analysed; the plot has been converted to monochrome for printing. The monochrome printing unfortunately makes the interpretation of the support strata interaction difficult, which is not the case with the colour coded maps.

![Figure 4 - TWAP (left) and number of yield events for July 10th- August 7th inclusive.](image)

The white regions in the TWAP plot indicate the regions where the sensor signals from the face were absent or so badly corrupted that no useful information could be extracted. The predominant cause of the white regions (>95%) is the simple absence of sensor signals.

The overloading of supports is indicated by the supports going into yield and continuing to yield without stabilisation throughout the support cycle. The number of yield events in a loading cycle is a good indicator of the intensity of overloading when cycles are all of similar time duration. Supports that have undergone yield events are also identified in the right hand plot of Figure 4. Some supports have had more than 10 yield events within the cycle.

Multiple yield cycles correspond to supports that continued to yield throughout the cycle and were therefore overloaded; i.e. they had inadequate capacity for the conditions. The number of yield events is influenced by the cycle time in addition to the intensity of loading, indicating that time is a critical component in assessing support-strata interaction on a longwall. The yielding patterns generally coincide with the high TWAP regions on Figure 4. In terms of strata support interaction there is a very big difference, however, between a heavily loaded support and an overloaded support. Assessing that the support is overloaded is not possible from TWAP alone.

The left of Figure 5 is a map of supports showing the set pressures lower than a user defined threshold and the right maps the cycle time for the shear. In this case, we have chosen 40% of yield as the threshold. This image is
intended to reveal the regions of the face in which set pressure is regarded as being too low for the conditions. The reason for the low set pressure may be that the set pressure could not be achieved due to roof damage and debris on the canopy under positive set regulation or that the overriding of positive set for the face and the use of manual setting for the face did not achieve the usual set pressures. Low set pressures often follow regions where supports have been heavily loaded, particularly if cycle times are long during periods of overloading. However, in some cases adequate set pressures can be achieved with extra effort and care. The software can be used to audit shield operation and allows Engineers to give feedback and guidance to operators. Prior to the development of the software this could only be done by very time consuming and tedious checking of the pressure-time record.

The multiple yield and low set pressure maps also show up faulty supports. For example at about leg 185 the leg is shown to be yielding every load cycle (refer to multiple yield map, Figure 4). An adjacent leg is shown to have a very low set pressure on every load cycle (low set map, Figure 5) indicating a fault. Minor roof control problems were encountered in this area of the face from time to time, particularly during long cycle times and towards the peak of the periodic weighting. However, a separate algorithm has been developed to automatically delineate faulty legs.

**Figure 5 - Supports having a set pressure below 180 bar (left) and cycle time (set to release) for the supports (right).**

The four images were used to deduce much of the support-strata interaction throughout the region where analyses were carried out. Feedback from mine personnel indicated that an accurate assessment of what had happened on the longwall face during the period of analysis could be made from these maps.

After mining about 45 shears periodic weighting can be observed on the TWAP and yield maps. There was some evidence of minor overloading of supports and in some cases low set pressures immediately following periods of high periodic weighting up to shear 210. Roof control problems with relatively minor delays were reported by the mine after these periods of support overloading. From about shear 210, the intensity of the periodic weighting started to increase. Overloading of a number of supports occurred in an approximately 3 hour cycle at around shear 210. There is evidence of low set pressures on both the TWAP and low set pressure maps after mining through this period of heavy weighting. Production delays due to roof control issues were reported by the mine.

Another relatively heavy loading occurred at around shear 242 where again a number of supports can be observed to be overloaded on the multiple yield map. Mining continued rapidly through this weighting event. A scattering of supports were observed to have too low a set pressure after this but far fewer than for the previous weighting event. Time was used effectively to negotiate successfully through this weighting event and achieving adequate set pressures in subsequent cycles also aided roof control.

A further weighting event where a number of supports became overloaded occurred around shear 256. There were two relatively long cycles of about 7 hours duration each during this weighting. It would be anticipated that cycles with a duration of about 7 hours could result in roof control problems when supports are so heavily loaded. Long cycles and data loss can be observed from about shear 260. Significant production delays were reported by the mine due to roof control problems.
Additionally, 29 legs on the face were identified by the software as having faulty yield or check valves during the period of analysis. Localised roof control problems were reported to occur periodically in the vicinity of some of these supports. More roof control problems were observed in the vicinity of these supports close to the peak of the periodic weighting and where cycle times were long.

The ability to identify supports that were being periodically overloaded rather than just heavily loaded, the identification of faulty legs and the set pressures being achieved for every load cycle, gave considerable insight to the mine staff as to the causes of the strata control problems that they had been experiencing and we were able to suggest mining strategies to mitigate the problems. Subsequently, when it was observed that support loading was intense, every effort was made to mine as quickly as possible, thus minimising the number of yield events and the amount of convergence. Every effort was also made to ensure adequate set pressures were maintained during subsequent shears during or after an overloading event. In this way the mine was able to subsequently successfully negotiate several overloading events, whereas previously significant delays to production had been experienced due to roof cavities. Identification of faulty support components reduced the incidence of localised roof control problems.

Although the “off-line” version of the code was useful in identifying the causes of roof control problems and understanding how to mitigate or avoid them, the real interest for the mine was an “on-line” version where analysis is possible in near real time. It is difficult from observations alone to differentiate between heavy loading and overloading, which supports have too low a set pressure for the conditions and which supports are faulty. For that reason the code was further developed in order to provide the required real time response.

**Real time analysis**

The switch to real time analysis from the off-line work that was done previously posed new challenges. The code that carries out the data analysis has been repackaged into libraries whose interface must be exposed to the graphical user interface (GUI) that the operator will use to configure the system and monitor the analysis results. The system relies on a server delivering data, principally from the longwall face computer, to a database server. A database application fetches blocks of data on a regular basis and passes them to the analysis routines which process the new information. The GUI also displays various forms of live data.

In one sense, the events on the longwall take place slowly; a longwall having one hundred supports is about 200 meters long and, depending on the thickness of the seam, cutting a web of coal requires somewhat less than one hour, if all goes well. However, the longwall is a busy place given that there are 200 cylinders to monitor and within which to maintain pressure and the actions of pushing the AFC or pulling up the support should be done as quickly as possible. It does not take many seconds to push the AFC and releasing, pulling and resetting the support can take place in a matter of seconds. To ensure that all events taking place on the longwall are logged as they happen a logging cycle every 20 seconds or faster is required. Joy and DBT longwalls have different methods of logging. The resulting flow of data, in addition to the control signals needed to keep the wall operating, is substantial, but manageable.

A schematic of the online system is shown in Figure 6. Data is collected from the face via a server that is provided by the longwall manufacturer. The data is stored in a database server, which is queried periodically by the GUI. Once sufficient new data are available for an analysis, the new set is sent to the ‘analysis engine’ for processing. On completion, a new result set is passed back to the graphical user interface for display in the familiar formats described above. The results are archived in the database for later retrieval.
The manner of presentation of the data to the longwall operators in real time follows the format that we found to be successful in our off-line code, with additions. Figure 7 is a screen dump of the control panel, showing TWAP and in the top right hand corner a map of the instantaneous pressures. Colour is used as a third dimension, as with our off-line code. All the different map types can be called up and historical data can also be presented. The instantaneous pressure map is a new development over the off-line code and gets over the problem of the fact that the analysed data is at least one shear behind because a load cycle needs to end before it can be analysed. This limitation is being addressed and with further developments analyses will be possible within the current shear before it ends. The method of presentation of the instantaneous pressures is shown in such a way that it is easy to interpret what is occurring on all supports across the face at a glance. The sloping blue lines for example are releases for the next load cycle. The cursor can be placed at any point on any of the maps and actual data, such as the pressure, are shown.

The system also displays a vertical colour scale which permits the translation of the colours to numerical values of the variable being displayed. In general, low values of a variable are cool colours with the higher values shown in hot colours. For example, pressures above about 400 bar will be a hot orange or red and high numbers of yield events will also be red. Once a user is familiar with the standard colour scales for each type of display, the colour bar can be turned off.

A report is generated of the percentage of time during a user defined period that identified problems (leaking leg, failing gauge, failed gauge, failed ram sensor) occurred; Figure 8.

A shearer production breakdown report is also generated for each shear (start and end time, duration of shear, percentage of total time productive, time cutting, flitting and parked); Figure 9.

Further developments are occurring to move from a Beta test version to a full release version of the software and extend its functionality so that it can be used on any longwall and analysis can be carried out in the current shear without waiting for it to be completed.

**CONCLUSIONS**

A load cycle characteristic concept has been developed aimed at quantifying longwall shield-strata interaction and has been encapsulated into off-line and on-line software. The concept has been validated by extensive analysis of about 700,000 support load cycles covering more than 2 km of mining advance at five different mines. The load cycle analysis concepts are a major breakthrough in understanding the interaction between a longwall shield and
the surrounding strata. Before these concepts were developed the pressure signals were largely unused, simply because of a poor understanding of what they meant in terms of support-strata interaction or for that matter the integrity of the support. The encapsulation of the concepts into software enables a rapid evaluation of the causes of

![Figure 8 - Report on potential faults on individual legs](image_url)

![Figure 9 - Screen dump of shearer production report](image_url)
roof control problems on a longwall face. The work has the potential to significantly reduce roof control problems on a longwall coal face by influencing:

- Shield design;
- Shield operation;
- Maintenance scheduling; and
- Panel layout and design.

Geotechnical model development would not have been possible without extensive and detailed analysis of support loading cycles (load-time curves between the setting of a support against the roof and release of the support immediately prior to its advance). Analysis was made possible through development of a new computer code that accepts the data stream from the longwall face (leg pressures, shearer position, DA ram position and equipment power draw) and accurately extracts loading cycles. This new code has proved far more accurate for load cycle identification than existing codes and also enables the critical load cycle features to be extracted that are essential for the accurate interpretation of support-strata interaction, the audit of shield operation and identification of faulty support components. The software has been extended from off-line to on-line in order to provide a real time response to changing strata conditions on a longwall. A Beta test version has been running successfully at BMA’s Broadmeadow mine for several months. The functionality of the code is being extended.

Shield loading is a complex interaction between: shield capacity and set pressure, the composition of the main and immediate roof, the presence or absence of leaking legs, extraction height, cycle time, panel width and seam depth. It has proven very difficult to isolate all these factors. The software developed by the authors that encapsulate their load cycle analysis theories now allows a rapid identification of the causes of shield loading and should facilitate a much better understanding and quantification of the different factors and is therefore a potentially powerful research tool.

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CRINUM MINE, 15 LONGWALLS 40 MILLION TONNES 45 ROOF FALLS- WHAT DID WE LEARN?

Dan A Payne

ABSTRACT: The Crinum mine is located near Emerald in the Bowen Basin of Queensland and began development in 1994 using 2 Joy 12CM30 continuous miners in the 3.4 m high Lilyvale seam. Longwall production began in 1997 and finished the 15th and final longwall in December 2007. Over the 10 years of longwall production 40 million tonnes have been extracted and the mine has experienced over 40 longwall face, 3 main gateend, and 2 tail gateend roof falls as well as 5 roadway roof falls away from the longwall. Weak roof (less than 10 MPa in the bolted horizon) has been the principal roof control issue at the mine. However, weak, highly cleated coal, water inflow from overlying aquifers, some minor structure and a diatreme in the main headings have also contributed to the challenges at the mine. This paper describes the geotechnical experience over the 10 years, the mine’s approach to addressing the issues and the relative success of these approaches.

INTRODUCTION

The Crinum Mine is a BHP Billiton Mitsubishi Alliance (BMA) underground longwall mine located 45km north of Emerald, Queensland (Figure 1). The mine began development in 1994 and longwall production in 1997 and finished the 15th and final longwall in December 2007. Crinum has mined over 40 million tonnes and experienced over 40 longwall face falls of ground that required reconsolidation. In addition, the mine experienced five maingate and three tailgate roof falls as well as five roadway roof falls away from the longwall (Figure 2). Weak roof (less than 10 MPa in the bolted horizon) was the principal roof control issue at the mine however weak friable coal, water inflow from overlying aquifers, some minor structure and a diatreme in the main headings have also contributed to the challenges at the mine. This paper will describe some geotechnical experiences at the mine with respect to development and primary support, secondary support, pillar design and longwall support. It should be noted that these are the experiences, explanations and solutions of and for the Crinum Mine and are not presented as being necessarily applicable to other sites.

EXPLORATION

Initially a 500 m square pattern of exploration drilling was carried out over the site. From this program over 150 core samples were tested for UCS. The sonic velocity of these samples was also recorded. A sonic velocity to UCS correlation was developed for the Crinum site. This correlation was applied to the bolted horizon of the mine and contoured over the workings (Figure 2). After about 5-6 longwalls it was recognized that the observations of bad ground conditions in the mine correlated closely with the <8-10 MPa contour on the derived UCS plot. Through further experience with the UCS contour and trials of drilling density, a surface exploration borehole spacing of 130 m down each gateroad prior to development was adopted as a procedural requirement for hazard plans (Figure 3). Since that time the UCS contour has been extremely accurate in planning and budgeting for the different bolting densities at the mine and predicting areas where cable bolting will be required and budgeting quantities.

This technique is further refined using the Roof Strength Index (RSI) developed at Kestrel which incorporates depth of cover and demonstrates that weak roof at greater than 150-180 m depth is much greater an issue than weak roof at less than 150-180 m depth.

The inability to carry out 3D seismic due to a layer of basalt near the surface or predict a couple full seam displacement faults with the already dense borehole spacing resulted in at least two changes to the mine plan when encountered underground.

PRIMARY SUPPORT

Full mesh versus W Straps

The mine began with a 6 bolts per metre pattern of 2.1 m fully encapsulated torque tension roof bolts installed through W straps. Even though this initial area of the mine was some of the best roof conditions; bedding, especially cross bedding, resulted in slabs of material falling between straps and injuring operators (Figure 4).
Within 10 pillars of entering the seam, full roof screen was employed. Subsequently the original straps have corroded and caused a hazard from dropping off the roof.

A 4 bolt per metre trial was carried out that may have been successful if it had been located in the best roof and shallowest section of the mine. Unfortunately the best roof area only existed in the initial mains area.

![Figure 1 - Location plan of the Crinum Mine](image1)

![Figure 2 - 2m UCS contour overlay onto the Crinum mine plan with the locations of roof falls also plotted](image2)

**Optimisation of the Manager's Support Rules**

When weak roof was encountered (below 10 MPa and as low as 3 MPa) several roof bolt pattern density increases were trialed. The first being to decrease the spacing per row to 0.75 m and when this was not successful, dropping the spacing down to 0.5 m. Even though this doubled the bolt density to 12 bolts per metre the roof continued to bag and lower between the bolt closest to the rib and the second roof bolt in the pattern, usually on the stress notched side of the road first. A trial of installing two extra bolts over the normal six bolt pattern between the outside bolt and the next bolt on each side of the roadway was implemented (there after called the 6:2 pattern, Figure 5). This bolt crossed the typical line of roof failure and was very effective in reducing roof movement. Of the
10 roof falls in roadways at Crinum mine, none occurred in a roadway with the 6:2 pattern installed. In fact, in one instance (MG8, B17-18), the roof began to deteriorate immediately behind the miner in development. The deputy invoked the 6:2 pattern for the next 50 m. Reports of deteriorating roof continued outbye and between shifts a 66 m long roof fall occurred trapping the continuous miner inbye. After two weeks of recovering the 66 m of fallen ground it was discovered that the fall had pulled up at the 6:2 pattern. The inbye lip of the fall demonstrated the failure line and mechanism (Figure 6). It was after this roof fall that a decision was made to rely on the hazard plan to trigger increases in bolting pattern densities rather than wait for roof movement triggers in the Strata Management Plan to invoke changes. The reliability of the UCS contour on the hazard plan eventually required the use of the 6:2 bolt pattern in 50-60% of the development drivage. This ultimately saved significant quantity of drivage time and material cost over the life of the mine had the more dense 0.5 m spacing been used or if a blanket 6:2 pattern had been adopted mine wide. A record drivage rate of 65 m in a 12hr shift using the 6 roof and 5 rib bolt per metre pattern, whereas a record of only 45 m in a 12 hr shift was the maximum achieved on the 6:2 pattern (8 roof bolts and 5 rib bolts per metre).

**Training**

Roof bolting is the most critical operation at any mine. If done correctly it provides stability for all other operators working under it. Early classroom training recognized the wide range of experience and understanding that operators had with roof bolting. Therefore a one hour long training DVD including actual video and video animation was developed to properly train operators in the theory of roof bolting and best practice installation.

Subsequently automated bolting rigs have decreased the variability in hole depth, penetration rate, spin time and hold time to nearly zero. Now every roof bolt is installed nearly identically and as close to manufacturer’s recommendations as possible.

**Encapsulation / Roof Bolt Tension**

The 2.1 m roof bolts installed in holes drilled with standard 27 mm win bits with a 1m two speedie resin have always been unencapsulated by 100-300 mm. This is mostly due to overdrilling/reaming of the hole to 28-29 mm and the fact that using the standard calculation for required resin quantity +25% is inaccurate for weak roof conditions. This remains the case to this day.
The tensioning of roof bolts at Crinum was initially verified through testing torque with torque wrenches after installation and quarterly roof bolt installation audits in which a torque-tension measurement was carried out. This was necessary as the breakout of the nuts was 108-122 Nm (80-90 lbft) and the Strata Management Plan required 203 Nm (150 lbft) torque on the nuts. The frequency of this check was increased to each bolting rig every 30m of advance by supplying two high torque test nuts in each supply pod. This continued until automated roof bolters were employed at Crinum at which time the entire stock of roof bolt breakout nuts was increased to 203 Nm (150 lbft) allowing a torque test on every roof bolt.

Experience at the mine suggests that tensioning of roof bolts in weak, highly laminated strata has some benefit to roof support and beam formation (approximately 10-15% improvement). However the act of tensioning performs a small pull test on the installation and ensures the plate is very tight against the immediate roof. It is believed that these two factors are at least as important as the traditional “clamping the beam together” theory. It is also believed that achieving full encapsulation would have increased benefit again.
Gloving / Overcoring

During the period of industry wide concern over gloving and unmixed resin, over coring was carried out at Crinum to assess the extent of the issue at site. At the same time some of the initial Hilti One Step bolts were being trialed at the mine. The overcoring showed no gloving or unmixed resin in the bolts overcored (although some had been observed in the goaf behind the longwall). Overcoring of the One Step bolts showed perfectly mixed resin (Figure 7). The overcore from both bolts showed that the roof material cored around the unencapsulated length broke up readily during the overcoring (lack of consolidation and confinement provided by the resin, Figure 8). This may indirectly demonstrate the benefit of fully encapsulating bolts. It also demonstrated the different borehole wall profile between the modified spade bit and the wing bit (Figure 9).

RIB BOLTING

With 3.4 m high roadways and highly cleated coal, the mine started with 2 x 1.2 m steel rib bolts and butterfly plates on both sides of the roadway in the main entries and the same with 2 x 1.2 m plastic cutable bolts in the block side. After reaching about 150 m depth it was realized that 3 improvements were required. Firstly, the removal of belt structure at the maingate was becoming risky due to sloughing of the rib in the longwall abutment zone. It was decided to increase the density of cutable bolts to 3 per metre after only 2 years of longwall mining. This improved conditions, however the observation of regular shear failures of the plastic bolts especially at depth resulted in a change to stronger fiberglass rib bolts for longwall 10. Also in line with the depth of cover of 150-180 m depth of cover and pillar side rib control issues at the maingate walkway and in the tailgate travel road, the installation of welded wire rib mesh on all pillars was adopted. This resulted in greatly improved rib performance, safety and tailgate travel road conditions (Figure 10).

Rib Bolt Encapsulation

Encapsulation has always been poor on the 1.2 m rib bolts using a 660 mm resin cartridge which consistently resulted in 300-500 mm unencapsulated. It is believed that this unencapsulated length contributed greatly to a wide range of rib failures including both tensile and shear failure of the bolts, rib spall around the bolt to the depth of encapsulation, nuts pulling through the plate, etc. and was more significant than the amount of tension applied to the rib bolts. Attempts to increase encapsulation by increasing resin quantity, or decreasing bit size always resulted in premature breakout or torsional damage to the rib bolt heads. The recent development of higher strength and breakout rib bolts has resulted in the ability to fully encapsulate rib bolts using 1 m resin capsules. This reduces the already simplified three types of resin at the mine (1m slow set for cable bolts, 660 mm fast set for rib, and 1 m two speedie for roof) to just 1m two speedie for both roof and rib and 1m slow set for cable bolts and has already shown improved rib conditions on development (Figure 11).

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Figure 7 - Good resin mixing and no gloving in standard roof bolts (left) and self drilling bolts (right)
Figure 8 - Broken overcore demonstrating lack of consolidation and confinement at the unencapsulated end of a standard roof bolt (bottom) and self drilling bolt (top)

Figure 9 - Cross section of borehole showing rifling achieved by standard wing bit
SECONDARY SUPPORT

Due to the requirement to maximize development rates to keep up with longwall retreat a practice of only installing enough roof support off the miner (6-8 roof bolts and 5 rib bolts per metre) to allow development to block out a pillar, and if secondary support was required it could be installed by contractors outbye. Due to the low bearing capacity of the roof 300 mm bearing plates were required on cable bolts as 200 mm plates would easily crush into the roof.

6 m vs 8 m Cable Bolts

The standard cable bolt length was 8 m based on initial consultant recommendations. However roof falls in roadways at Crinum have been the roughly triangular shape (Figure 12), the height of which has always been within 400 mm of the width of roadway (4.8-5.2 m high). Using the 4-6 m horizon above the natural arch shape of the roof failure as being secure, 6 m cable bolts installed at 1/3 the way across the roadways, have been employed in up to 50% of cable bolt installations, but only when hazard plans showed good anchorage in the 4-6 m horizon (Figure 13).

Post Grouted vs Point Anchor Cables

After an initial practice of installing passive cement anchored cables, the mine employed resin point anchor post tensionable cable bolts as the standard secondary support. This practice was successful in developing in, and taking the longwall through, some very weak ground. Although conditions looked bad, with a large amount of roof...
lowering, no maingate roof falls occurred in cable bolted ground. When post grouting cables became popular, this practice was adopted at Crinum. Unfortunately two roof falls occurred in the maingate over the BSL in areas that had post grouted cables installed (both below 180 m depth of cover).

The main issues identified were:

- Crinum installs cables outbye, therefore if the roof has already delaminated, the failure horizon has been predetermined.
- The grout encapsulates the cable and therefore if the roof loads it at one horizon (the predetermined failure horizon), the roof only needs to move a small amount to take the cable past its ultimate strength as there is very little elongation available.
- As the cable is fully grouted there is no sign of weight on the bearing plate and together with the minimal movement required to fail the cable the deputies were not able to identify imminent danger.
- The abutment loading of the longwall is seen as somewhat of an unstoppable force. Some roof movement will occur in weak roof.

A decision was made to revert back to point anchor cables in gateroads only. Crinum had never experienced a maingate fall in cable bolted ground for the first 10 longwalls, experienced two consecutive maingate falls on longwalls 11 and 12 using post grouted cables (Figure 14) and then no falls since on the final three longwall gateroads.

Beside the ability to properly visualize roof deterioration with point anchored cables and allow significant roof movement before failure, the installation of post groutable cables allows the subsequent use of the grout tubes to inject PUR into the already existing cables (and roof fractures) instead of having to try to wet drill around the congested area of the BSL and install extra cables or inject PUR when the roof is already in bad shape at the gate end. Effectively it allows for a fourth line of action response. This has had to be actioned only two times at Crinum in the maingate intersection but potentially prevented roof falls on both occasions.

Spiling through the gate end falls along the roofline with about 14 HQ drill rods from outbye the 10-14 m long fall to up over the maingate canopies and installing steel sets underneath after chipping out the fallen material appeared to be the best and was the only method used to recover the three maingate falls at the mine. Limited material had to be removed, limited damage was done to the BSL and no false roof had to be reestablished.
Cable Bolted Beam

The policy of not grouting cables was *not* applied to installation roadways. The intention on installation roads has been to build a roof beam and minimize the amount of roof movement and thereby maintain as much inherent strength as possible and has been the standard since LW6.

The concept of building a strong thick beam has been taken to an extreme at the new Crinum East mine where a thick layer of extremely weak roof (3 MPa) exists at the 6 m horizon and above (Figure 15). Historically 8 m and 10 m cable bolts were employed on installation roads at Crinum with height of softening extending to 5-6 m. Due to the difficulty in drilling and anchoring in the 6-10 m horizon, the design methodology of forming a thick beam was followed. 6m long, 80 t cable bolts were installed on a dense pattern, pretensioned to 40 tonne and post grouted. Although the face road was widened to 7.8 m for line shields and up to 9.8 m wide for gateends and shearer stable, minimal additional roof movement occurred in the 6 m beam.
Figure 15 - Geomechanical roof section showing 6 m cable bolt horizon for LW16 installation road

Top Down Versus Bottom Up

The argument over bottom up versus top down grouting of cables was demonstrated in one particular application at the mine. An install road (LW6) was driven first pass and experienced a range from about 25 mm at the gate ends to greater than 300 mm roof movement near mid face on first pass of a 4.8 m wide roadway. A roof support plan of cable bolts and trusses was designed and included bottom up grouting of the cables. The installation road cables took pallet after pallet of microfine cement especially in the area of greatest roof movement with the cement migrating into all the open bedding planes. It was reported that pumping was discontinued at one point due to the sound of cracking in the roof and gurgling and sloshing of the wet cement in the open bedding planes in the roof. When it came time for widening, the roof which had the greatest amount of movement first pass experienced the least amount of movement second pass. In fact any areas which had greater than 90 mm of roof movement experienced very little roof movement second pass and areas with less than 90 mm experienced significant roof movement (Figure 16). It was postulated that this phenomenon had two causes. Firstly, the grout was of a greater strength than the initial roof material itself and created stronger beds of significant thickness. Secondly, as roof fails it opens up bedding planes and slides along bedding planes. The rate of roof movement increases exponentially as the fractures increase and the frictional resistance is reduced by the newly created openings. Refilling those voids with grout reestablishes the frictional contacts on the bedding planes and provides a bulk material which has to go through the failure process again. As the secondary support has already been installed, the reconsolidated roof “mass” performs better than the initial roof. PUR injection into the roof works in the same way with the greatly added benefit of adhering to the bedding plane contacts and being even stronger. This experience is qualified in that top down grouting may be more applicable in moderate roof that has little initial movement and maintaining the integrity and inherent strength of the beam itself is the goal.

High Strength - High Tension Cable Bolts

An ACARP assessment of high strength, high tension cable bolts was carried out in a particularly weak area where development was being slowed by excessive roof movement. The A heading (travel road) of Maingate 7 required cable bolting after every about 40 m of drivage, resulting in the machine being fitted to the belt road and “B” heading being driven just slightly behind A heading. The B heading didn’t require cable bolting which was assumed to be due to the stress relieve caused by A heading being driven first. This was further demonstrated when the 80 tonne cables in A heading were finally tensioned to 40 tonnes and subsequently the B heading became unstable and required a full complement of cable bolts to regain stability. It was a result of this trial and the constant problems maintaining stability on installation roads that 80 tonne cables with 40 tonne tension became the standard support on installation roads.
Figure 16 - Roof movement first pass and second pass on the longwall 6 installation road demonstrating
the reduction in roof lowering in the midface area where the most microfine cement was pumped into the
roof.

Corrosion

It was recognized after five years of cable bolting in the main entries that post grouting in life of mine entries (+5
yrs) was applicable. This was demonstrated by approximately 35 cable bolt failures in an area of 3 MPa roof which
continued to creep over the life of the mine. The failed cables showed excessive corrosion and would have
benefited greatly from being fully grouted with a corrosion inhibiting grout.

PUR Injection

The Crinum mine was one of the largest consumers of PUR in Australian coal mines for two years, exceeding 300 t
both years. This was primarily for longwall falls but also included installation roads and other outbye roads.

Bolted Roof Injection

When injecting in roadways a procedure was setup which included the following:

- Standing support (usually propsetters) was required to be installed in the roadway prior to
  injection
- Pump pressures were limited to 85 bar to reduce the risk of "jacking" the roof down.
- Electronic roof to floor convergence monitoring was employed and roof movement limited to 8 mm
  in any one hole injected
- No inexperienced operators were allowed under the unstable ground
- The pump was located outside the unstable area
- Low or no expansion PUR was used
- The ability to pump more than 200 kg in a single hole not intersecting a coal seam was of great
  benefit
- Roof extensometry was reestablished after the PUR had set and before the props were removed

Longwall Roof Fall Injection

Injection on longwall roof falls was initially under the control of PUR contractors. However after initial experience,
detailed PUR support and injection plans were developed by the geotechnical engineer including number, depths
and angles of holes, injection limitations, use of roof bolts, cable bolts, dowels or spiles and sequence of injection
(Figure 17). Injection quantities were required to be submitted at the end of each shift and plotted on the support
plan for migration.

Lessons learned at the Crinum mine included:
Migration occurs for a maximum of 4 m ahead of the face in coal and therefore coal holes were reduced to 5 m to minimize drilling time.

Very low expansion PUR is most effective in coal

When trying to secure faults or fractures in weak ground, drilling through and installing steel bars or dowels is critical (this applies to cementitious grout injection as well)

12m is about the maximum distance possible to fully PUR encapsulate a splice or dowel

Solid bars are better as spiles than flexible cables

When trying to preconsolidate a fault or weak ground the following experience was gained.

Drilling greater than 50 m requires full directional drilling capabilities to ensure adequate accuracy

Spiling weak ground requires a density of about one splice per metre

Due to its brittle nature and lack of adhesion consolidating by injecting cementitious grout into broken ground is prone to longwall abutment reactivating the fractures and negating the consolidation effect. The use of steel in these holes increases their effectiveness ten fold.

There is a huge tendency for the cement injection personnel to water down the cement mix in order to gain pumpability. A ratio of 1:1 water:cement will usually equate to about 12 MPa strength on microfine cements.

LONGWALL FALLS

During the two years of highest PUR consumption the mine experienced very weak roof areas (< 5 MPa for up to 20 m above the seam) on two different longwalls. On Longwall 7 the mine experienced 19 consecutive roof falls over the final 140 m of the longwall panel while on Longwall 9, 15 consecutive roof falls occurred (Figure 2). These falls occurred as soon as the longwall had mined past the influence of the previous PUR reconsolidation.

The learnings from these falls included:

The number of events provided operators and managers with the experience that trying to “catch the lip” almost always resulted in a larger fall and longer recovery and therefore the decision to pull up and PUR was eventually made at the first sign of trouble and therefore longwall triggers were reacted to much sooner.

Less than 5 MPa roof for a height of greater than 4 m above the seam is extremely difficult to control even with good mining standards

Longwall shields must be in excellent condition to control weak roof as it is extremely unforgiving

When injecting PUR into fractured weak roof always include steel dowels, cables or bars. Weak roof will simply rip away from the PUR leaving sheets of PUR with a thin coating of the host rock attached to it (Figure 18). The steel provides a tensile resistance to the reopening of the fracture.

When trying to reestablish longwall production during a roof fall in which the wall will continue to mine in weak or fractured ground, spiling is an effective means of providing a false canopy.

Several times Crinum installed 6 m and 12 m dowels horizontally just above the roofline ahead of the face, at least one if not two per shield (Figure 17), and injected them with PUR (not for the purpose of consolidation but for encapsulation and anchorage). The false roof allowed the entire canopy of the shields to get under solid roof, reestablish adequate set pressures, clear any stone remaining on the face and get the longwall back to full production before leaving the zone of consolidation thereby giving it half a chance of maintaining stability and production rates.

Weak roof combined with weak friable coal is particularly prone to roof falls when production slows due to delays or preparation for longwall takeoff. It has become standard procedure to preinject the coal seam with PUR ahead of the face when coming to takeoff in weak roof.

Cavity Fill

It is accepted practice that when a roof cavity gets to a certain size on a longwall, cavity filling becomes a requirement. The material provides confinement to walls and roof of the fall to prevent the cavity from growing provides the shields with an ability to achieve some set pressure and provides protection for equipment and operators. However it also provides some assistance in the goaf. If the longwall has loaded out a lot of stone, that stone has not reported to the goaf and therefore there is more room for roof material and further caving. It is possible that this allows the goaf to progress further ahead of the face.

Although a more expensive material, the phenolic foams are much quicker and require less high risk shuttering than standard cementitious foams, thereby making them cost effective (Figure 19).
Figure 17 – PUR injection – longwall recovery support plan

Figure 18 - Side and end view of PUR which had been injected into a fracture in a longwall roof fall and then ripped back out during remobilization of abutment stresses demonstrating the greater adhesive strength of the PUR than the weak rock itself.
Figure 19 - Cementitious cavity fill material (left) versus Phenolic foam (right) showing the reduced amount of shuttering required.

Figure 20 - 3D plot of roof fall cavity measurement using survey instrumentation

A learning at Crinum was how important it is to get an accurate estimate of the volume of the cavity required to be filled. Contractors and suppliers are good at looking at a cavity and saying it will require X amount of product and then if it requires more explaining it by saying the cavity was bigger than we thought, however on two occasions during roadway fall rehabilitation, Crinum was able to measure the exact volume of the cavity by using measuring poles and survey cavity measuring equipment (Figure 20). Using the expansion factor of the foam cement quoted by the supplier, which makes the product appear cost effective, these cavities should have only require a quoted amount of material (including the dense low expansion layer at the bottom of the cavity). However it was discovered during the application that nearly twice the quoted amount of material was used. This could be attributed directly to improper mixing and/or product performance. This process can be quality assured by accurately measuring the volume of the cavity and regular and random sampling and testing of the delivered product at the nozzle.
PILLARS

Criniu gatesoad pillars are 30 m wide rib to rib down to a depth of 220 m. This gives a UNSW pillar design Factor of Safety of about 1.4 which is rated as a probability of failure of about 2 in 100. At 3.4 m high and 200 m deep, Criniu pillars were showing signs of significant load and were demonstrating that they were close to their design limit. However as it was only the initial pillars in each longwall (the panels mined up hill) that were at this depth it was decided to leave the pillars at 30 m solid and install some additional secondary roof and rib support in the deepest pillars to increase the stability in this localized area. This double rib bolting and 50% overlapping additional mesh provided the necessary confinement to ensure stability and was an effective means of maximizing the resource. The benefit of this rib bolting pattern had been quantified in a previous ACARP funded rib support trial. Pillars in Criniu East will be fanned to a width of 35 m where they go down to 250 m depth of cover. The increased width will improve stability and minimize water and gas connectivity from previous goafs.

Instrumentation

Sonic Probes

Initially the mine used sonic probe extensometers to perform detailed roof monitoring. This resulted in valuable information to validate the initial roof support and mine design model. However it was realized that when the mine reached the routine production phase, sonic probe extensometers had the following limitations:

- The probes were fragile and easily damaged
- The probes were expensive to repair or replace
- Readings from one probe to another or to a repaired probe are not the same and require a recalibration of readings.
- The readings were not directly interpretable underground at the site and therefore triggers could not be acted on until readings were delivered to surface and input into a computer

Routine Monitoring

The routine monitoring and trigger response stage of the mine was better served using mechanical telltale at every standard intersection and four point electronic extensometers at critical areas such as installation roads and takeoff roadway intersections. The electronic telltale provided an accuracy of greater than 0.1 mm and immediately interpretable readings underground. It was also realized that responding to instrumentation was dependent on frequency of reading and speed of data input and therefore a procedure was set up such that all telltale and resistive potentiometer extensometers would be read by ERZ controllers and the data entered by the control room operator on shift. This included outbye telltale read by outbye ERZ Controllers on a schedule similar to stone dust and bag samples.

A further improvement was made to the traditional overlapping tube style telltale by converting to the clock it style telltale. This enabled an improved accuracy of about 0.5 mm and discontinued the practice of assuming the reading hadn’t changed or guessing at the reading when a ladder wasn’t available or was too much trouble to carry around, which was necessary in a seam height of 3.4 m with the old style telltale. An improvement to this style of telltale would be a clear plastic cover over the dials and further corrosion resistance.

A limitation of mechanical and electronic extensometers is their susceptibility to corrosion if water is present. Some initiatives were attempted to reduce this problem with little success. More successful was a regular inspection of readings and instrumentation to ensure it is still functional.

TARPS

Roof movement TARPS at the mine have always been L1, 10 mm in either horizon, L2, 20 mm in either horizon and level 3, 40 mm in either horizon. At mid mine life a mine wide telltale results review was carried out. This review showed 50% of the telltale readings (either horizon) showed less than 10mm of roof movement, 15% less than 20 mm, only 5% between 20 and 40 mm and 30% showed more than 40 mm of roof movement (Figure 21). In addition several plots were made of rates of roof movement showing acceleration points at 10 mm and 20 mm. These results validated the trigger levels set showing that below 10 mm the roof was stable and required no further action, between 10 and 20 mm it needed a more frequent inspection, at 20 mm it needed a higher level inspection and plan for secondary support because if it made it past 20 mm without secondary support it would continue to 40 mm after which a roof fall was imminent.
Blast Monitoring

Blast monitoring for vibration from open cut blasts can be very helpful in understanding the effects on underground workings. The application of 1000 m exclusion zone is not applicable for underground as the underground environment is not exposed to fly rock hazards. After sufficient monitoring of vibration from open cut blasts a graph can be developed which predicts the expected vibration (peak particle velocity (ppv)) from any individual blast based on Maximum Instantaneous Charge (MIC), distance and type of blast (pre split, overburden, pre strip, etc). Values below 2 ppv are difficult to sense by operators especially if there is any noise or activity underground. 2-10 ppv (especially 7-10 ppv) are perceivable by sound and vibration and operators should be notified of the plan and expected time of the blast. Values 10-21 ppv are loud, can create a wave of imbalance if standing on a platform, cause rattling of steel in a crib room and displace bits of dust and rib along roadways. Figure 22 shows results from the monitoring of an open cut blast within 700 m of underground workings.
LONGWALL SUPPORT

7 day Operation

When longwall mining started in 1997 Crinum was a 5 day a week operation. Very often, after the weekend downtime, a roof fall or significant roof slapping would develop on the first few shears Monday. By 1999 the mine switched to 7 day mining which solved this recurring problem and reduce it to after major equipment delays.

Monitoring

In 1999, Crinum commissioned the GeoGuard system of longwall shield monitoring. That system ran successfully, albeit with its own limitations, for six years. It demonstrated that no significant weightings were occurring at the Crinum mine as shown in Figure 23. If there was a caving interval it was about 10 m which was not sufficient to generate excessive loads. GeoGuard was also used to monitor shield condition, maintenance, set pressures, etc. Routine audits of longwall support performance included plots from GeoGuard which showed shields which required priority maintenance and these audits were submitted to longwall maintenance for action.

Maintenance

A general learning was that after seven longwalls shield maintenance became critical. Staging (blipper) valves became clogged with debris and often didn’t operate correctly causing the leg pressure to operate on the smaller upper cylinder reducing the set and yield force applied by the shield by 30-40%. This was initially recognized by the lower cylinder gradually climbing up to its full extension. Once at full extension the set force of the shield is reduced even further as the lower leg comes in contact with the steel of its outer housing. Yield valves were discovered to eventually yield at a much lower pressure as the orifice which controls the yield pressure scourrs out after years of yielding. Faulty check valves can limit the shield pressure to line pressure only. Pin slop in the linkages can allow significant horizontal movement before resistance is applied to the roof by the canopy depending on what point in the “S” curve the canopy is set to the roof. In addition seals began to leak. Often shield electronics will make decisions based on the pressure in the MG leg. The tailgate leg pressure is irrelevant.

Support Design

It was eventually recognised that the shields at Crinum had the following design limitations:

1. The flippers (coal deflectors) have a designed rotation of only 90° to deflect coal from the 3.4 m seam and prevent it from toppling across the panline and spill plate to the walkway. If they had been designed to today’s double-knuckle 180° ability it is believed that some roof falls could have been prevented and operator protection would be improved during roof fall recovery and longwall bolt up. Because the face spalled immediately after cutting and generally buckled at the stone bands (in fact the shearer at Crinum is more of a loader and has to cut very little intact coal) the 90° coal deflectors frequently could not contact the coal face even when double chocked (Figure 24).
2. It was recognized that Crinum mine would be mining in weak roof conditions during the initial mine design and shield specification. Therefore the canopy was designed long to extend out ahead. In fact, when double chocked the canopy extends 0.57 m into the next web of the shearer. However due to the 3.3 m seam height, there was insufficient room to make a rear walkway. This meant that a walkway had to be maintained in front of the legs of the shields. The long forward canopy together with the minimum walkway results in a poor canopy ratio and a poor tip loading of the shields.
3. It is believed that having the shield legs closer to the face would have provided better support and a better canopy ratio, however there are dimensional limitations on how this could have been achieved.
Longwall Support Improvements

The following improvements were made at Crinum to improve shield performance in addition to the routine maintenance program:

1. Routine shield pressure monitoring was initiated
2. Longwall support audits were carried out by the geotechnical engineer
3. A dedicated high set pump system was installed
4. A study of the hydraulic fluid delivery system was carried out and the hydraulic supply lines were increased in size and the filter sled reengineered to reduced restrictions.
5. A shield leg changeout program was initiated
6. 19 spare shields were purchased in order to allow a future face extension but also enable a changeout and rebuild of worn shields at each longwall move prior to the face extension (enough to rebuild every shield).
7. A yield valve change out program was initiated
8. The control system was changed
9. A full retrofit is being carried out prior to installation in the new Crinum East Mine

After a tailgate fall which occurred between 915 mm diameter tin cans spaced at 5 m (in which a stress window on offset longwall start positions contributed), the mine adopted a blanket support plan of 915 mm diameter cans on 3 m centres. After a period of monitoring and some staged trials this pattern was eventually dropped to 700 mm diameter cans on 3 m centres in areas of weak roof or at a depth greater than 180 m (Figure 25). Standing support has always been installed in the maingate travel road behind the wall to prevent the requirement of installation in the dust of the subsequent operating wall. Although this limits clearance for access to the tailgate end of the wall (which the 700 mm cans improved equipment access up to a 913 loader) it allows early loading of the standing support. Roof extensometers revealed that peak roof movement in the travel road occurred 50 m behind the face and continued until 270 m behind the face. The practice of trying to keep the cans even with the wallface allowed improved conditions in the tailgate and standing support that was well and truly set to the roof before tailgate abutment approached.

Bolt up for longwall shield recovery was switched from high grade AX roof bolts to low grade mild steel bolts which reduced the snapping off of roof bolt heads by the force of the shield canopies. This had no negative effect on roof support during takeoff.

A trial of rib sprays was carried out and it was determined that any stiff highly reflective shotcrete could be used in workshops where low stress and no change in stress was to occur. However for longwall abutment areas these stiff products cracked and fell off in dangerous slabs. A more flexible product worked reasonably well on longwall takeoffs in combination with friction bolts where face spall had previously been a big problem using polymer grid mesh and resin rib bolts (Figure 26).
After several years of "dry" mining the Crinum longwalls began to mine beneath a tertiary flow of water bearing basalt. Longwalls 7 and 8 showed signs of increased water make on the seals at the back end of the wall (lowest elevation). Both these walls mined within 90-80 m interburden to the aquifer with a progressively thinner layer of clay below the basalt. This is consistent with other mines in the Bowen Basin with 90m being the start of water percolation at other sites. However Longwall 9 mined to less than 70 m interburden (Figure 27) at a low point in the bowl shaped aquifer and experienced an inrush which was initially estimated at 120 L/s and settled to 75 L/s. Without an adequate pumping system in place this resulted in unacceptable pressures to build on the back end seals and these seals were subsequently opened to release the pressure and flood the installation road of longwall 10 (eventually causing a discontinuity in production during the longwall transfer).

The mine had initially been setup to handle water make from the goaf of the first few longwalls. However after several years and panels of "dry" mining this set up had not been advanced with the mine workings. The learning was to always be aware of the distance to overlying aquifers and have adequate pumping systems in place to handle any potential water make.

**WATER INFLOW**
Flooding and then Reestablishing Workings

In addition to learning the limit of interburden distance, several things were learned about the flooding of an installation road. The Longwall 10 installation road had already been widened and was flooded to a depth of 8 m above the roofline, which included five pillars out the main and tailgate. Install roads in this area were notorious for requiring roof rehabilitation usually in the form of PUR injection. Depth of cover was 200 m. A strange wave action existed at the waters edge which was never fully understood but was postulated to have been created by the action of pumping the water, differential pressure between the MG and TG or the operation of an M20 air pump which was discovered to have been operating underwater. Due to the weak roof and rib many persons at site believed the faceroad would certainly collapse and be unusable and a new faceroad would be required. The expectation was so high that life jackets (Figure 28) were required for anyone accessing the edge of the water in anticipation of a rush of water caused by a fall of ground on the face road. Actual observations and learning included:

- The entire faceline roof stayed intact with some small areas of local flaking
- The ribline was in excellent condition
- Although flooded to 8 m deep an air pocket was trapped on the roof of the faceroad and a 100 m section never came in direct contact with water (Figure 28)
- The roof areas which did come in contact with water (when open extensometers holes were inspected) the water only penetrated a maximum of 200-300 mm into the roof strata.
- The action of the waves on the roofline scoured out any bagging roof and the preexisting gutter in the maingate was washed out to reveal a height of failure of 1 m along the chain pillar side (Figure 29).
- The action of the waves caused the broken roof material which had been sitting on the mesh to wear through the mesh and form hundreds of perfect 100 mm x 100 mm square and bedding plane thick pieces of rock. (Figure 30)
- The action of the waves washed a clay band out of the seam and caused an up to 50mm gap between the top of the pillar and the roof line for a significant distance over the pillar from the roadways. This didn’t cause any problems for longwall mining.

Figure 27 - Interburden isopach showing thickness of Permian strata between the seam and a tertiary aquifer
Figure 28 - Water level 4 pillars outbye installation road in the tailgate (left). Installation road after being pumped out and showing the water line where a pocket of air was trapped for 100 m

Figure 29 - Pre existing roof bagging which was washed out by the action of the water
DIATREME

The dense borehole spacing at the mine did not predetermine the existence of a diatreme which was present in the main entry trunk conveyor roadway. Development began to observe the coal seam become sugary with an increase in mineralization. Then with little other transitional indications a 70 m by 30 m mass of broken, semi consolidated material was intersected (Figure 31). Although a few diatremes had been encountered in NSW no others were known to exist in the Bowen Basin. It was learned that roof bolt anchorage was beam formation was not possible and the zone required steel sets complete with concreted inverts to successfully mine through it. Although thought to be bad luck for the only diatreme known to exist in the Bowen Basin to occur in the main trunk conveyor road at the mine, its or another diatreme existence in the longwall panel would have been worse.

CONCLUSIONS

The Crinum mine successfully operated for ten years under less than favourable ground conditions, generally producing within the top five longwall mines in the country. This was achieved by adapting to and developing procedures for dealing with these difficult conditions. Numerous learning were realized during the life of the mine, not all of which were in line with traditional beliefs.

There is a saying “We will know exactly the best way to mine and support this ground by the time the mine closes”. The Crinum mine was almost there, but not quite.

Figure 30 - Examples of roof material which eroded through 100 mm x 100 mm welded wire roof mesh during the action of the water.

Figure 31 - Broken semiconsolidated material of the diatreme (left) and its contact boundary with the coal seam (right).
EFFECT OF GROUTING ON LONGWALL MINING THROUGH FAULTS

Terry Medhurst\textsuperscript{1}, Michael Bartlett\textsuperscript{2} and Renate Sliwa\textsuperscript{3}

ABSTRACT: The demand for increased production, safety and resource recovery has put pressure on the coal industry to find methods to mine through fault zones. One of the processes used to reduce the likelihood of face instability is to consolidate the strata around the fault using grouting techniques. Since grouting techniques are being used more frequently in practice, the industry requires a greater understanding of the effect of fault consolidation on ground improvement and associated strata response during mining.

This paper presents the results of a recent ACARP research project aimed to assess faulted areas to determine the need or otherwise for grouting and their likely impact on mining performance. A review of past fault consolidation projects was undertaken to determine their “success” in longwall operations and to identify factors that influence longwall ground control.

This paper includes a comparative analysis of grouting results from several mines. The outcomes will provide guidance to assist operators in understanding when fault grouting is required, how it might be implemented and expected outcomes of the grouting program.

INTRODUCTION

A risk assessment of longwall retreat through a fault zone will quickly identify potential financial losses or personal injury due to strata failure as major risks. Smith (2006) suggests that major causes and contributing factors of strata failure and/or poor longwall operating conditions are commonly:

- Poor horizon control
- Poor face alignment
- Incorrect setting of shields
- Stopping or inconsistent face retreat
- Breakdowns or failure of equipment to perform
- Crews cutting inconsistently or with low morale
- Poor cutting sequence
- Inability to react to problems
- Mining into ground that is outside the capability of the equipment or people

A fault management strategy that includes the following critical controls provides the greatest likelihood of a safe and consistent longwall retreat through a fault zone:

- Accurate and precise geological knowledge of the nature of the faulting and lithology
- Knowledge of the geomechanical behaviour of the strata, including any likely beam effects, particularly with a coal roof
- Adherence to basic longwall operating standards, including horizon control, cutting sequence, creep control and shield setting
- A proactive maintenance regime targeting the section of the face likely to be most affected by the fault zone
- An effective water management strategy
- An effective circle of communication between geologists, geotechnical engineers, the longwall department and underground crews, with an agreed plan of attack

Often in good ground conditions, the absence of one or more of these controls may be tolerated with little effect on production. However, during retreat in faulted ground, the likelihood of a significant interruption to face operations is greatly increased. It is therefore essential that, when mining into faulted areas, all of these controls are in place and any of the root causes of strata failure or poor operating conditions are addressed.

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A research project funded by the Australian Coal Association Research Program (ACARP) was initiated to investigate issues associated with mining through faults (ACARP C13015, 2006). This project focused on a detailed monitoring program of mining through a fault at an Australian longwall mine. Analysis of the fault pre-consolidation program including comment on grout takes, pressures and penetration was. Longwall support monitoring and face stability analysis was also used to examine overall strata-support interaction processes.

Following the detailed study, a review of past fault consolidation projects from various mines was undertaken. The aim of the review was to devise a means to assess faulted areas to determine the need or otherwise for consolidation and their likely impact on mining performance.

**FAULT CONSOLIDATION USING GROUTING**

**Current Grouting Practices**

The civil engineering industry has developed several techniques for permeation grouting and grout design for use in dam construction, civil works and tunnel sealing. The coal mining industry has adopted many of these methods and attempted to apply them to fault consolidation on longwall mines.

The conventional method of grouting has been substantially developed over the last 40 years from empirical results obtained from different grouting activities. Houlsby (1990) has extensive experience in the area and has reported his findings in a practical and applicable manner.

After drilling the grout holes and water testing to determine the permeability, a decision on the grout type, initial water:grout ratio and maximum grouting pressure is made. The initial water:grout ratio is guided by the results obtained from the water tests starting with a thin grout mix and slowly increasing the grout water cement (w/c) ratio throughout the grouting process.

A review of different mine sites shows the most common grouting method involves drilling grout holes at a given spacing to intersect the fault whilst minimizing the drill lengths and maximizing the drill angle. The grout is then pumped into the total length of the grout hole until significant increases in pressures are achieved. If the grouting pressures don’t increase over time the grout mix is thickened and the thicker grout is pumped into the grout hole. The backpressure is monitored using a bleeder hole, which can also be used to pump grout into should any blockages in the grout hole occur.

The selection of grout can be wide and varied depending on the application and the results required from the grouting project. The grout type chosen for the task is primarily a function of the aperture of the joints in the fault and the associated costs. Water cement ratios can vary from about 0.3:1 by weight to around 10:1, however research by Houlsby (1990) and Weaver (1991) have shown grout mixtures with w:c greater than 5:1 have little strength and durability. Grout mixtures have also become more sophisticated in recent years by using complex admixtures and microfine cements. Modifiers such as superplasticers allow enhanced grout strengths by providing greater penetrability at lower w:c ratios.

**Drill Holes, Spacing and Layout**

Boreholes can be drilled inseam or from the surface depending on the depth of mining, the ground conditions and the available equipment. Both methods are used in Australia for fault consolidation work. The general consensus for sealing rock in the grouting industry recommends borehole spacings of 1 to 3 m and row spacings of 1.5 to 2.5 m (Kutzner, 1996).

For inseam holes, borehole spacing is typically between 2.5 m and 4 m. The drill patterns commonly adopted generally aim to target the first 1 m to 2 m of immediate roof above the longwall face. Figure 1 shows a plan view of a typical inseam drill pattern. For surface holes, a 5 m x 5 m grid pattern up to a 5 m x 10 m grid pattern is typically used around the proposed fault location.

**Grout Injection Pressures**

Injection pressures within the grouting industry can vary significantly, ranging from 1 bar to 50 bar depending on the application and the pumping equipment available. In general the extent of grout spread is proportional to the grouting pressure, the extent of fractures, and inversely proportional to the cohesion of the fluid grout. Even though little evidence is available to give a standard grout pressure range, particularly in coal mining, the easiest method to determine the maximum grouting pressure is to keep the grouting pressure below the cracking pressure (Kutzner, 1996).

The cracking pressure is dependent on the rock conditions, however a simple calculation for the maximum grout pressure is based on the unit weight of the overburden relative to depth. A more detailed approach considers the potential for borehole fracture based on the in-situ stress state and rock properties. In general terms, hydrofrac stress measurements indicate that the magnitude of minimum horizontal stress is equal to the shut-in pressure, or
the minimum pressure needed to keep a fracture open after pumping has been stopped. A review of hydrofrac stress data in the Bowen and Sydney basins indicate that the minimum total stress in coal seams generally varies between 50% and 85% of overburden pressure at typical mining depths (Enever et al, 1999).

Figure 1 - Typical drill hole layout into LW block

Measuring Grouting Outcomes

Grouting has traditionally been used for seepage reduction in tunnelling and the dam construction industry and collectively this work forms the basis for measuring grouting success. In general, a reduction in permeability or Lugeon value (1 Lugeon unit = 1 litre/metre/minute at 1000 kpa) to less than 2 is often considered successful as a maximum leakage criterion. The cost of grouting to reduce the Lugeon values below 2 is often considered uneconomical.

A method for measuring grouting success for longwall operations is less evident since the primary purpose is to consolidate the faulted rock mass. In general, the characteristics of the fault(s) and surrounding strata govern longwall face stability. However, methods of measuring the pre- and post-mining effects of ground improvement are difficult to quantify.

Faults that intersect one or two longwall supports do cause significant delays. Therefore one measure is that if the fault consolidation could confine instabilities to one or two supports then it could be considered sufficient for consolidation, having reduced delays to an acceptable level (ACARP C10019, 2003). Similarly, if the influence of grouting can be related to the level of ground improvement, eg via permeability testing, this may provide another means of measuring grouting success.

One approach is to simply maximize the volume of grout injected in to the fault(s). This is the current fallback position used by the industry given the lack of alternative approaches. A database that contains records of the volume of grout injected, hole location, the type of faulting encountered, mining performance and related test data should provide a starting point to quantify the influence of grouting.

FAULT TYPES AND GROUND CONDITIONS

Fault Stability Criteria

In general, major structures present a high level of risk if they are orientated at less than 20 degrees to the longwall retreat line. This is due to the alignment with goaf cracks and mining induced fractures, and the length of the longwall face that is exposed to poor ground conditions at any one time.
Using simple mechanics, the most unfavourable orientation of a shear plane in relation to the major stress direction is about 30°. Faults dipping at 60° to 90° towards the longwall face would therefore generally present the greatest risk of instability when subjected to vertical abutment loads, as shown in Figure 2. Similarly, faults oriented 30° to the longwall panel would generally be most prone to instability as a result of horizontal abutment loads, as shown in Figure 3.

Lee (1966) outlines a number of factors likely to cause fault reactivation and subsidence based on U.K. experience as a result of longwall mining:

- The fault must dip over the panel and toward the panel centre with the panel in the footwall of the faults, i.e. towards the longwall face
- The fault surface expression must be about 0.2 times the depth towards the goaf from the gateroads.
- A longer fault is more prone to reactivation than a shorter fault. Also a fault that does not completely cross a panel and extend well beyond its limits, is less prone to reactivation.

The first point highlights that shown in Figure 3, in which fault reactivation is more likely when the overlying material is able to “slide” towards the goaf. This can be further exacerbated by the presence of multiple structures, which can often form wedges. Wedge failure can be particularly damaging if low angle (thrust) structures are present.

The second point suggests that the fault must be in the maximum tensile strain area of the subsidence trough between the gateroads and the trough centre. The third point provides that the fault completely crosses the panel. This is logical because the end of the fault provides a restraint against reactivation by the unfaulted strata, requiring shearing through unaffected strata to continue lateral fault movement.

From a mining standpoint it is generally preferred to first intercept the fault in the tailgate. This is preferred over the alternative, in which first interception at the maingate would result in sustained tailgate damage as mining progresses. Similarly, faults oriented near parallel to the face present an extreme risk of instability and therefore need to be approached at a compromised angle (say 30°) to minimise face exposure at any given time whilst limiting the overall length of retreat in the fault system.

The potential for fault movements in the “high risk” zones is overprinted by the local fault characteristics, for example an open, soft structure versus a tight structure. The combination of open voids and potentially elevated abutment loads in a high risk zone can therefore present a challenging set of conditions for longwall mining. Unfortunately, longwall operating and production requirements necessitate mining through faults at angles that are generally the most unfavourable from a fault stability standpoint. This is why the correct choice of mining horizon is so important and needs to be coordinated with accurate determination of fault characteristics and targeting of stabilisation measures.

**Water Pressure Testing**

Water pressure testing is perhaps the simplest and certainly most widely used method for assessing the need for grouting. Houlsby (1990) recommends that while conducting a water pressure test the pressure should be held constant at one bar for 15 minutes with the water take measured in 5 minute intervals. This is recommended to get a representative set of data for the WPT. Analysis of the water pressure test data can give an indication of the
permeability but also the characteristics of the fault. Decreasing Lugeon values indicate voids being filled whereas increasing Lugeon values indicate either void creation by washing out the hole or under certain conditions or hydrofracing of the borehole.

As discussed previously, grouting has traditionally been used for seepage reduction in tunnelling and the dam construction industry and thus seepage mitigation is the basis for measuring grouting success. The current study has provided evidence that permeability testing can provide a good indication of fault conditions and the need for grouting. The grouting industry has developed a guide to grouting requirements on the basis of rock mass permeability. For example,

- 1 Lugeon unit is the type of permeability where grouting is hardly necessary.
- 10 Lugeons warrants grouting for most seepage reduction jobs.
- 100 Lugeons occurs in heavily jointed sites with relatively open joints or in sparsely cracked foundations where joints are very wide open.

Analysis of permeability test data suggests similar limits apply to longwall mining. In general, faults with a hydraulic conductivity in excess of 10 Lugeons are likely to take significant amounts of grout and will benefit from a grouting program. For tighter structures, there is some evidence to suggest that grouting may be less influential on longwall face stability depending on the fault orientation and panel loading influences, particularly for Lugeon values less than about 2.

Lugeon tests should be carried out over a standard length to ensure consistency of results. The location of packers and subsequent calculation of Lugeon values can dramatically influence estimate values. In particular, mine operators should ensure that a good standard of testing is undertaken and directly targets the fault structure(s). The tests should also be supported by good drilling records that provide as much detail of the fault structure as possible. In general, where a large length of borehole is pressured, the Lugeon value provides a measure of the mean permeability of the formation. For short test lengths, there is likely to be a much closer correlation between Lugeon values and the real permeability of the formation, particularly for faults.

**Influence of Grout**

It is generally accepted that pre-grouting can improve the quality of the rock mass. Barton et al (2001) states the reasons for increased rock mass quality and easier excavation are due to any or all of the following:

- Filling of joints and voids
- Closing of secondary joints
- Glueing and strengthening of the parts of the rock mass
- Reduction of water flow
If any or all of the above factors are achieved, an inherent outcome of the grouting process is an increase in stiffness of the rock mass. This is a very important but often overlooked result for fault pre-consolidation activities in longwall mining. For example, void filling using grout injection would only provide a minor strength increase in the fault (binding effect) but can increase the stiffness of the fault by an order of magnitude. Following the mechanisms shown in Figures 2 and 3, it can assumed that if the fault is stiffened to a level similar to the surrounding strata then stress transfer through the fault system will be improved thereby minimising stress concentrations in critical areas.

Soft structures such as fault zones are often low stress zones and are unable to transfer stress both in both the vertical and horizontal. Stress is redistributed, which means that elevated stress magnitudes (can be 20 % higher) and localised changes in orientation can develop, typically 10 – 20 m either side of the fault(s). Under elevated stress conditions the strata support interaction characteristics can change significantly. An additional 20 % abutment load can equate to an additional 50 m depth of cover. This can change the roof damage profile considerably. Figure 4 shows a potential roof damage profile under high stress and soft roof conditions.

In general, two approaches can be taken for grout injection depending on the size, style and orientation of faulting. These are:

- Specific targeting and grouting/reinforcing of the fault zone itself, or
- Grout injection and reinforcing of the fault and overlying roof beam

Specific targeting of the fault zone can be achieved with either in-seam or surface holes and would generally be expected to provide similar results in softer ‘unconfined’ fault systems, eg normal fault with large throw and broken infill. A general indication of grout coverage is shown in Figure 5.
The stiffness and competency of the immediate roof strata adjacent to the fault(s) is also important, as is the properties of the fault planes. The softer or more broken the surrounding strata, the more likely benefit to be gained from grout injection. Conversely, grout injected into a clean normal fault that intersects a series of strong sandstones for example, is likely to provide minimal impact on stability.

The stiffer the fault system, the less effective grout injection is likely to be. A typical stiff fault system might be a more complex array of small conjugate faults that produce a blocky rock mass. Under these conditions reinforcing of the fault and overlying roof beam would be the preferable option as it also helps to minimise wedge type failures. The use of grout injection from surface based drilling will generally provide a greater spread of grout higher into the strata than would be achieved from in seam drilling. This former is the best method for developing a thicker 'reinforced' roof beam.

The correct choice of mining horizon is the most critical factor in managing face stability. This requires a balance between choosing a stable roof profile and maintaining practical longwall operating tolerances. In general, the bearing pressure of longwall support canopies is diminished 1 m or more into the roof (Medhurst, 2005). It is therefore important when choosing a mining horizon to minimize the thickness of weak roof strata and/or ensure that the pre-consolidation effort has been directed to provide at least a 2 m thick competent roof beam.

ANALYSIS OF FAULT GROUTING

Grout Volumes and Pressures

Thirteen cases of fault pre-consolidation grouting in underground longwall mines were analysed. Where possible, data was collected that included fault characteristics, water test data, grout volumes, grout pressures, drilling patterns, spiling and observations during mining. Total injected grout volumes ranged from 4,000 litres up to 60,000 litres for the various cases.

Various grout mixes were used, however all cases in the database can be regarded as microfine or ultrafine products. Particle size for the grouts varied with a maximum of 40 microns, but most report a maximum size of 12 microns. Similarly, drilling method and hole size varied across the dataset, some drilled from surface, others in-seam intersecting the fault(s) and inseam drilled sub-parallel to the fault(s).

Volumes of grout take were calculated to measure the effectiveness of grouting. To calculate the grout takes the volume of drill hole void space and spiles was subtracted from each grout volume for the corresponding grout hole.

A broad correlation between grout pressure and grout take could be observed across the entire grout database, shown in Figure 6. Given that a “grouting to refusal” approach was used, the maximum pressure was used in the analysis. Significant grout take occurs when injection pressures exceed about 2500 to 3000 kPa. This suggests that perhaps at these pressures, hole fracturing of the strata during injection may begin to develop, which facilitates the increase in grout take.

Clearly, the “tightness” of the fault(s), the hole spacing, grout mix and injection pressure will all contribute to the overall grout take. Simple correlation between grout takes and pressures do not consider the characteristics of the strata or fault system that is being treated. Detailed analysis indicated that grout takes appeared to be greater in the larger throw faults than the smaller faults.

Geological Controls on Grout Take

A comparison between grout take and maximum fault throw is shown in Figure 7. All of the grouted structures are segments of larger normal faults, which have moderate to steep dips. Some faults are part of more complex horst or graben structures, and three examples were related to step-over zones. Fault throws range from 1 m to 7.8 m.

The data shows that grout take increases for faults with greater than 4 m throw. However, changes in grout injection pressure can also account for this increase. Therefore, an attempt was made to estimate the overall trace length of the fault system intersected by the drill pattern. From this estimate, a simple measure of maximum grout take along the faults could also be made for comparative purposes, Figure 8. This assumes that all available grout is taken up in the fault system.
Figure 6 - Relationship between grout pressure and grout take

Figure 7 - Comparison of grout take and maximum fault throws

Figure 8 - Comparison between fault throw and grout take in fault
The data shows a general increase in grout takes with the maximum throw of the fault. A similar trend also prevails in terms of injection pressure, Figure 9. The general conclusion is that the main controls on grout take are influenced by the style of faulting and the injection pressure used during grouting. In particular, the larger throw faults were also likely to be subjected to some later reactivation, which is thought to have contributed to the amount of grout take. Although data is limited, permeability test data for these cases also suggested that these faults were more ‘open’ than the smaller throw examples.

![Figure 9 - Relationship between injection pressure and grout take in fault](image)

**APPROACH TO GROUTING FAULTS**

**Drill Program and Testing**

The aim of the drill hole grout pattern is to intersect as many joints and bedded planes in the fault as possible while minimising the drill lengths and maximising the intersection angle. This implies a compromise between the costs associated with drilling and the effectiveness of grouting based on the drill hole angle and frequency of intersections with the fault. Since several mines have reported different levels of success using different drilling patterns and techniques, there is no hard rule about which drill design is the best. Drill hole design in reality is site specific and involves many factors that are controlled by the geological features of the fault such as location, size, orientation and depth of cover.

Along with the initial exploration drilling, the first assessments are usually based on seismic profiling and can often provide a broad measure of fault throw. Based on the available data, if fault throw is greater than 4 m, it is likely that roof and longwall face stability would benefit from a drilling, grouting and spiling program. Often the need for such measures however is less obvious, and in this case it is recommended that targeted drilling and permeability testing program be undertaken to investigate fault characteristics.

Water testing will provide a measure of the ‘openness’ of the fault(s) as well as provide some measure of potential grouting requirements. It is also important to note that water testing over short lengths (less than 5 m) will provide a more representative measure of fault permeability than if undertaken over long hole lengths. Current data suggests that in areas where Lugeon values are less than about 1 or 2 ground conditions may permit longwall mining. An example of the relationship between grout take and permeability is shown in Figure 10.

**Estimation of Grouting Requirements**

The conventional approach of “grout till refusal” is governed by the grout mix properties and the applied pressure during grouting. Moreover, it is generally thought that the selection of grout parameters can greatly influence the amount of grout take, and is reliant on operator skill. Interestingly, despite the importance of the grout mix, a wide variation in the reporting of grout mix parameters was found. In some cases, grout volumes were reported in kilograms of cement, while others in litres of grout pumped.

Clearly both the volume of grout pumped and the proportion/amount of cement need to be reported as a means to determine effective grout strength/quality. The final cement content in the overall grout volume shows a general trend of a 2:1 mix design across the database, Figure 11. In general, provided the grouting contractor can
demonstrate good quality control procedures and adhere to the design controls, our review of case studies suggests that grout takes are more influenced by fault characteristics and injection pressures rather than small differences in grouting parameters.

![Graph showing the relationship between permeability and grout take.](image)

**Figure 10 - Typical relationship between permeability and grout take**

![Graph showing the comparison between grout volume and cement used.](image)

**Figure 11 - Comparison between grout volume and cement used**

In general, the gate-ends and the last third of the face towards the tailgate are the most critical areas for ground improvement. In large throw faults, stability is likely to be improved using pre-consolidation measures. In other cases, a focus on stabilising gate-ends and other high tensile zones along the face may be sufficient. A summary of general grouting requirements is provided in Table 1.

In rare cases very difficult ground control problems have been encountered whilst longwall mining through faults. In hind-site it may have preferable to relocate the longwall around the fault. These cases usually have possessed three main characteristics:

1. Faults oriented 30° or less to the longwall panel, and
2. Faults dipping at 60° to 90° towards the longwall face, and
3. Fault is first intercepted at gate-end in high stress zone.
As previously discussed, it is generally preferred to first intercept the fault in the tailgate. This is preferred over the alternative, in which first interception at the maingate would result in sustained tailgate damage as mining progresses. The problem can be further exacerbated if longwall panels are oriented such that a stress concentration develops along the maingate side.

### Table 1 - Minimum suggested requirements for fault grouting in longwall mining

<table>
<thead>
<tr>
<th>Fault System Description</th>
<th>Fracture Frequency</th>
<th>Permeability</th>
<th>Stabilization Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Complex fault system oriented near parallel (&lt;30°) to and dipping (60° to 90°) towards longwall face</td>
<td>N/A</td>
<td>N/A</td>
<td>Review potential for severe mining conditions based on detailed study of fault characteristics</td>
</tr>
<tr>
<td>Complex fault system with at least 2 faults and maximum throw greater than 4 m. Some minor thrusts suggesting reactivation.</td>
<td>N/A</td>
<td>N/A</td>
<td>Comprehensive pre-consolidation and reinforcement program</td>
</tr>
<tr>
<td>Complex step-over zone in normal fault system with maximum throw greater than 4 m.</td>
<td>&lt; 5 m</td>
<td>&gt; 2 Lugeons</td>
<td>Comprehensive pre-consolidation and reinforcement program</td>
</tr>
<tr>
<td>Complex step-over zone in normal fault system with maximum throw greater than 4 m.</td>
<td>&gt; 5 m</td>
<td>N/A</td>
<td>Target grouting at gate-ends, high tensile zones and high permeability (&gt; 2 Lugeons) zones</td>
</tr>
<tr>
<td>Complex fault system with at least 2 faults and maximum throw less than 4 m.</td>
<td>&lt; 5 m</td>
<td>&gt; 2 Lugeons</td>
<td>Comprehensive pre-consolidation and reinforcement program</td>
</tr>
<tr>
<td>Complex fault system with at least 2 faults and maximum throw less than 4 m.</td>
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<td>N/A</td>
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<td>N/A</td>
<td>Target grouting at gate-ends, high tensile zones and high permeability (&gt; 2 Lugeons) zones</td>
</tr>
<tr>
<td>Single normal fault with maximum throw greater than 4 m and continuous across panel</td>
<td>N/A</td>
<td>&gt; 2 Lugeons</td>
<td>Comprehensive pre-consolidation and reinforcement program</td>
</tr>
<tr>
<td>Single normal fault with maximum throw greater than 4 m and terminates in panel</td>
<td>N/A</td>
<td>N/A</td>
<td>Target grouting at gate-ends, high tensile zones and high permeability (&gt; 2 Lugeons) zones</td>
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</tr>
<tr>
<td>Single normal fault with maximum throw less than 4 m and terminates in panel</td>
<td>N/A</td>
<td>&gt; 2 Lugeons</td>
<td>Stabilize gate-ends using PUR and additional roof/rib support. Stabilize unstable longwall face during mining.</td>
</tr>
<tr>
<td>Single normal fault with maximum throw less than 4 m and continuous across panel</td>
<td>N/A</td>
<td>&lt; 2 Lugeons</td>
<td>Stabilize unstable longwall face during mining.</td>
</tr>
<tr>
<td>Single normal fault with maximum throw less than 4 m and terminates in panel</td>
<td>N/A</td>
<td>&lt; 2 Lugeons</td>
<td>Target poor rib areas using PUR and additional roof/rib support. Stabilize unstable longwall face during mining.</td>
</tr>
</tbody>
</table>
CONCLUSION

Thirteen cases of fault pre-consolidation grouting in underground mines were analysed. A strong correlation between grout takes and injection pressure was found across all sites. A simple measure of maximum grout take along the faults was also made for comparative purposes. By estimating the overall trace length of the fault system intersected by the drill pattern, the volume of grout take per metre length of fault could be calculated. The data shows a general increase in grout takes with the maximum throw of the fault and that the overall grout take increases significantly for faults with greater than 4 m throw.

The influence of grout type and mix design parameters is often quoted as a most critical parameter in successful grouting. However, the grout mixes used in the case studies were generally similar.

Overall, it was found that the main benefits of grouting were void filling, producing an increase in stiffness of the rock mass. If a fault is stiffened to a level similar to the surrounding strata then stress transfer through the fault system will be improved thereby minimising stress concentrations (and failure) in critical areas. The stiffness and competency of the immediate roof strata adjacent to the fault(s) is also important. It follows that the softer or more broken the surrounding strata, the more likely benefit will be gained from grout injection.

The correct choice of mining horizon is the most critical factor in managing face stability. This requires a balance between choosing a stable roof profile and maintaining practical longwall operating tolerances. It is therefore important when choosing a mining horizon to minimize the thickness of weak roof strata and/or ensure that the pre-consolidation effort has been directed to provide at least a 2 m thick competent roof beam.

The orientation of fault zones in relation to the direction of longwall retreat plays a significant role in determining grade control, and the sections of the face that will be affected by faulting. In general, major structures present a high level of risk if they are orientated at less than 30° to longwall retreat. Faults dipping at 60° to 90° towards the longwall face would also generally present the greatest risk of instability when subjected to face abutment loads.

ACKNOWLEDGMENT

The assistance provided by ACARP is kindly acknowledged. The collaboration of the various mines that provided data and in particular Anglo Coal Australia is greatly appreciated. Similarly, grouting contractors Groutech Pty Ltd and Eastern Mining Services Pty Ltd (Multigrout) were most helpful in technical discussions and data gathering. Mr Greg Smith is also acknowledged for his assistance in the project.

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GEOLOGICAL AND GEOTECHNICAL INFLUENCES ON THE CAVEABILITY AND DRAWABILITY OF TOP COAL IN LONGWALL TOP COAL CAVING MINING

Patrick Humphries\textsuperscript{1} and Brett Poulsen\textsuperscript{1}

ABSTRACT: Longwall Top Coal Caving (LTCC) is a means of efficiently mining thick (>4.5m) coal seams and is an established technology in China with more than 20 years experience and over 100 faces in operation in a variety of different mining conditions.

A CSIRO – ACARP funded project has utilised the database of experience gathered by the Chinese to develop a LTCC caving assessment procedure for evaluating Australian coal seams based on numerical modelling.

The CSIRO developed continuum code COSFLOW has been used to assess LTCC mining at multiple scales. COSFLOW analyses the global stress redistribution from the ground surface to below the mining seam examining the influence of geology and the geotechnical properties of the rock mass and allows for an initial assessment of LTCC based on mining depth, coal strength and seam thickness early in the development of a thick seam mining project.

INTRODUCTION

Longwall Top Coal Caving (LTCC) is a means of efficiently mining thick (>4.5 m) coal seams, it is an established technology in China with more than 20 years experience and over 100 faces in operation producing over 200 Mt. The successful introduction of the LTCC mining method into Australia will require the reduction in risks, both financial and to personnel, associated with this mining technique. This reduction in risk can be achieved through careful consideration and assessment of coal seam and overburden characteristics, the selection and design of appropriate mining equipment and the control of gas, dust and spontaneous combustion.

CSIRO funded by ACARP has undertaken two major studies - C11040 and C13018, investigating the suitability and application of LTCC in Australia and the development of engineering design tools to classify and categorise Australia’s thick coal seams.

Longwall top coal caving employs both coal cutting of the lower portion of the coal seam accompanied by caving and reclamation of the ‘top’ coal at the rear of the supports. Coal is first cut from the longwall face using a conventional shearer and Armoured Face Conveyor (AFC) arrangement working under hydraulic face supports that incorporate a rear coal conveyor and rear cantilever / flipper arrangement. Face cutting heights are generally in the range of 2.8 to 3.0 m to maximise the coal left for caving. As the support is advanced forward after the shear the rear conveyor remains in place in preparation for the caving sequence. The caving sequence allows the broken coal above and at the rear of the supports to flow from the goaf onto the rear conveyor and through to the gate end transfer. This flow of coal onto the rear conveyor is controlled by retracting the rear cantilevers of selected supports exposing the rear conveyor to the goaf coal which ‘caves’ into the free space. Once an area has been caved, the rear cantilever is extended back out into the goaf stopping any further influx of goaf material. The caving process may be repeated at the same position (secondary caving) if further coal is present before the rear conveyor is finally advanced forward under the rear of the support ready for the next shearer cycle.

Figure 1 shows the general arrangement of an LTCC face.

Depending on the conditions in the mine, various caving sequences are employed to maximise the top coal recovery. In many cases the top coal caving is the primary production mechanism rather than coal cutting by the shearer, and overall face cycle times depend entirely on caving rates rather than shearing rates.

With this in mind, gaining a fundamental understanding of the theory and principles behind the caving process and the importance of coal strength and vertical stress relationships cannot be underestimated. To achieve a successful application, it is useful to first study the Chinese coal fields and translate and apply this experience to Australian conditions.
A CSIRO – ACARP funded project has utilised the database of experience gathered by the Chinese to develop a LTCC caving assessment procedure for evaluating Australian coal seams. Regression analysis of a range of parameters has identified depth of mining, coal strength and seam thickness as primary factors influencing coal caveability & drawability.

Potential sites for LTCC may be identified by assessing the caveability and drawability of top coal (the amount recovered by the rear conveyor) from the above three parameters that can be obtained from bore holes early in the exploration program and mining lease appraisal. As knowledge of the coal environment is obtained, the model may be expanded and refined to account for additional parameters influencing LTCC mining.

One of the major risks of LTCC is that the top coal either doesn’t cave or caves behind the rear conveyor and is lost in the goaf. Poor drawability of the top coal implies the caved coal is in fragments too large to flow onto the rear conveyor and results in lost coal or excessive downtime causing AFC overloading.

Scientific studies and detailed investigation is required to determine a particular site's potential for LTCC. To reach this point we must first understand the theory of Top Coal Caving and apply it according to Australian conditions before moving forward and assessing that particular site’s potential.

**TOP COAL FRACTURING PROCESS**

The process of fracturing and crack evolution in the top coal is critical to the success of LTCC and is dependent on abutment pressure and coal mass strength (Zhongming et al 1999). Poor fracturing will cause larger blocks to form and poor caving through the rear AFC will result. Excessive fracturing will in turn cause roof control issues ahead of the face supports. Top coal fracturing occurs through shear failure and tensile cracking. The fracturing process begins ahead of the LTCC face when vertical stresses in the coal seam increases due to its abutment with the excavated panel. The top coal undergoes horizontal dilation as it is loaded vertically with little or no horizontal confinement upon it entering the caving zone as shown in Figure 2 before finally caving at the rear of the LTCC supports. Estimation, through modelling, of the degree of fracturing occurring during this cycle is at the core of predicting LTCC production. The top coal fracturing process can be separated into four stages or zones as shown in Figure 2.

1. Deformation zone

This zone is located ahead of the peak vertical stress, the amount of compression is small and deformation is mostly elastic.

2. Compression fracturing zone.

This zone is located in between the peak vertical stress (ie front abutment) and the coal face, typically a distance of around 10 to 15 metres. Horizontal dilation of coal is greater than vertical in this stage.
3. Loosening zone.

This zone is located above the rear of the LTCC supports. The top coal in this zone is broken up by the action of repeatedly loading and unloading the face as it retreats. Vertical displacement is larger than horizontal displacement especially in the upper top coal.

4. Caving zone

This zone is located at the rear of the canopy of the face supports. Coal in the bottom portion of this zone is broken into small blocks and easily drawn. The upper portion of the top coal is often compressed into an arch and is drawn by articulating the rear caving door or by advancing the supports.

Figure 2 - Top coal fracturing zones

THE CAVING PROCESS

Top coal caves because it has been fractured due to abutment stresses and loosened by the mining process (the lowering and setting of supports) as outlined previously to the extent that when the longwall chock is advanced, removing the lower restraint from the top coal (in the caving zone) directly above it, overburden pressure and gravity induces the broken coal to flow down onto the rear AFC. The cave, or flow, of coal may require some external stimulation from ‘feathering’ with the rear caving door but once initiated, the top coal will cave back to a given angle above the supports (the caving angle) dependent on its strength. Hard coals may have a caving angle of only 40 to 70 degrees where as soft coals may have a caving angle up to 100 to 110 degrees. Figure 3 shows the measurement of caving angle.

Figure 3 - LTCC caving angle
Current understanding of the interaction between in situ and mining induced abutment stresses, coal strength and overburden deformation during the LTCC mining process comes from 20 years of observation in Chinese mines and from extensive physical analogue studies and numerical simulations.

Top coal caves during the LTCC mining process if the interaction of stress, overburden deformation and chock movement is sufficient to exceed the strength of the top coal and induce new fractures and loosen the natural fractures of bedding and cleat throughout the top coal thickness to enable sufficient caving. Creating the optimal block size distribution in the top coal allows for maximum recovery (ie having a high percentage of coal blocks created in the top coal that can cave onto the rear conveyor).

**PARAMETRIC STUDIES OF INFLUENCING FACTORS**

**Chinese Parameter Study**

Chinese experience with LTCC has identified depth of mining, coal strength, top coal thickness, stone band thickness, degree of coal fracture and immediate roof thickness as parameters influencing caveability in a LTCC operation. A numerical study presented in the book Theory and Technology in Top Coal Caving Mining (Professor Jin Zhongming 2001) recently translated by CSIRO EM undertakes a systematic analysis of these parameters and by regression develops a formula for the caveability and drawability of top coal presented as:

\[
y = 0.704 + 0.0006338 H - 0.00786 Rc + 0.238 C - 0.1797 Mj + 0.01434 Md \quad [1]
\]

Where:

- \( H \) is depth of mining (m)
- \( Rc \) is the UCS coal strength (MPa)
- \( C \) is a coal fracture index
- \( Mj \) is stone band thickness (m)
- \( Md \) is top coal thickness (m)

The relative importance of these parameters based on the results of scientific studies is \( H, Rc, Mj, Md \) and \( C \). Immediate roof thickness is known to influence caveability in practice however it was shown to have little effect in numerical modelling studies completed by the Chinese.

From a study of 23 LTCC faces in China (Zhongming 2001) the caveability index \( (y) \) was shown to be linearly related to the seam recovery ratio as is presented in Table 1.

**Table 1 - Relationship between Chinese caving index ‘y’ and success of LTCC**

<table>
<thead>
<tr>
<th>LTCC Classification</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mining Conditions</strong></td>
<td>Very good</td>
<td>Good</td>
<td>Medium</td>
<td>Bad</td>
<td>Very bad</td>
</tr>
<tr>
<td><strong>Caving Index (y)</strong></td>
<td>&gt; 0.9</td>
<td>0.8 – 0.9</td>
<td>0.7 – 0.8</td>
<td>0.6 – 0.7</td>
<td>&lt; 0.6</td>
</tr>
<tr>
<td><strong>Top coal Recovery (%)</strong></td>
<td>&gt; 80</td>
<td>65 – 80</td>
<td>50 – 65</td>
<td>30 – 50</td>
<td>&lt; 30</td>
</tr>
</tbody>
</table>

**CSIRO Parameter Study**

CSIRO undertook a parameter study using its own in house numerical modelling code COSFLOW and investigated through detailed modelling the following parameters based on Chinese research and investigation.

**Depth of mining (vertical stress)**

Chinese research suggests the magnitude of the front stress abutment will ultimately influence the caveability and drawability of top coal. In turn the stress abutment will consist of the in situ stress usually linearly related to the cover depth and an additional amount due to mining that can vary from 1.5 to 5 times the in situ level.

**Horizontal stress**

Not considered highly important in general by the Chinese. However, the higher horizontal stress regime evident in Australian coal fields are known to influence the general overburden deformation process and hence may influence the forces acting locally on the top coal. The top coal is influenced by the overburden within the fractured zone of the roof, which could be expected to be significantly horizontally de-stressed.
Coal strength and natural coal fractures

Coal strength will determine the damage induced by the abutment stress on the top coal. Natural fractures in the coal including bedding and cleat will assist in loosening the coal and may be activated by either the abutment stress or the loading and unloading action of the chock.

Thickness of top coal and inter seam stone bands

The thickness of top coal influences the success of LTCC mining in several areas. Chinese experience suggests LTCC mining is suitable in seams from 4.5 m to approximately 12 m in thickness, greater than this the coal may cave but at an angle of break such that it falls behind the rear conveyer or that the flow of coal is choked off due to the flow characteristics of the fragmented coal resulting in poor drawability. Thicker seams may have a beneficial influence on the stress abutment increment due to the greater deformation of the overburden however if the seam is too thick the coal may be damaged only in the top section of the seam resulting in poor caveability due to the immediate coal above the supports still being relatively intact.

Location, thickness and strength of inter seam stone bands will have a generally negative influence on the caveability and drawability of top coal and together with dilution, inter seam stone bands will have a detrimental influence on the success of LTCC.

Overburden properties

Strength and thickness of the immediate roof and overburden in general will influence the front stress abutment, the compressive deformation of the top coal and the force directly transmitted to the seam at the free face of the goaf.

Chock capacity

Unlike conventional longwall mining where the trend in recent years to ensure face stability has been towards stronger and stiffer chocks, the LTCC process benefits from a lower capacity support (around 600 tonnes) and the cyclic lowering and raising the canopy during face advance and from chock closure during the cutting cycle. These actions open fractures on bedding planes and induce the second fracture set drawn in oblique to fractures induced from abutment stresses. LTCC chocks are also not subject to as intense periodic weighting effects due to the thickness of the coal roof they interact with and hence may be of lower capacity than those used under a hard roof.

COSFLOW MODEL OF LTCC CREATED FOR PARAMETRIC STUDIES

The COSFLOW model of LTCC has been formulated as a plain strain model representing a section on the mid-line of a panel that extends from the surface to 200 m below the mining seam. Analyses are undertaken by removing elements from the lower 3 m portion of the coal seam, supporting the top coal by a representation of a LTCC chock and removing the top coal at the rear of the chock allowing the overburden to deform and form the ‘goaf’ as shown in Figure 4. The excavation and support sequence is modelled in 2 m increments from an initial undisturbed state to a state representing 500 m of mining at which stage the results are extracted.

A generalised representation of a typical overburden is used in this study with an immediate roof of 5 m thickness and main roof of 20 m thickness is shown in Figure 5. This model was used for the parametric studies undertaken in the ACARP project.
Figure 4 - COSFLOW model for LTCC showing face support (chock) and elements of the numerical mesh. Mining progresses from left to right with face coal removed, chock advanced and top coal removed forming goaf.

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>final cover</td>
<td>50.0</td>
</tr>
<tr>
<td>main roof 2</td>
<td>variable</td>
</tr>
<tr>
<td>main roof 1</td>
<td>20.0</td>
</tr>
<tr>
<td>immediate roof</td>
<td>5.0</td>
</tr>
<tr>
<td>top coal</td>
<td>3.0-9.0</td>
</tr>
<tr>
<td>coal</td>
<td>3.0</td>
</tr>
<tr>
<td>floor</td>
<td>5.0</td>
</tr>
<tr>
<td>base</td>
<td>200.0</td>
</tr>
</tbody>
</table>

Figure 5 - Strata represented in COSFLOW model of LTCC
COSFLOW LTCC Stress Path

From an initial pre-mining stress state, the top coal destined to cave into the rear conveyer (or not and be lost to the goaf) undergoes a complex stress path that opens and loosens the natural defects of bedding, cleat and joints of the coal or introduces new compressive fractures. The stress path includes:

- Increased vertical loading from the front stress abutment with good confinement. The front stress abutment may increase one and a half to five times the initial vertical pre-mining stress level while maintaining the horizontal confinement. Such conditions may approximate a triaxial tests and the damage in the form of compressive yield may be induced.

- Reduced vertical confinement due to the advancement of the longwall face supports. A ‘block’ of top coal may be subject to up to seven cycles of the chocks each reducing then increasing the vertical confinement. This may open any horizontal bedding or cleating in the coal.

- Reduced horizontal confinement approaching the goaf. As the top coal approaches the free face of the goaf the reduced horizontal confinement may allow vertical bedding or cleats to loosen. In addition, the deformation of the overburden will load the top coal from above.

It is the accumulated damage from this stress path that allows the top coal, when freed of the confinement of the chock, to cave under gravity into the rear conveyer.

An indicator to reflect the modelling results related to the caveability and drawability of the top coal is required for the parametric study. In the analyses reported here, the average horizontal plastic strain in the elements defining the top coal (ie above the chock) is used as it reflects the movement of damaged coal towards the goaf, the greater the movement towards the goaf, the better will be the caving.

Results are averaged over the final 80 m of mining to account for any natural fluctuations (ie from periodic caving) resulting from the deformation of the roof strata.

Parametric Study Results and Analysis

Both Chinese and Australian studies identify depth of mining and coal strength as the two most important factors (in that order) influencing the caveability of top coal in LTCC mining. Increasing depth of mining increases the absolute value of the abutment stress resulting in increased damage in the top coal and hence better caving.

Conversely, the studies suggest, and are backed up by observation, that increased coal strength negatively influences LTCC caveability and results in larger coal block size negatively influencing the drawability of the caved coal.

The influence of the insitu horizontal stress was not quantified however qualitatively it was found that increasing the horizontal stress from one to two times the vertical stress reduced the damage in the top coal. A plot of the normalised parameters examined in the study verse the measure of damage (horizontal strain) is presented in Figure 6.

![Figure 6 - Results on a common figure by normalising the parameters. Normalising the parameter is done by dividing the variation of the parameter by the base case value. Hence for mining depth where the base case is 300 m, 150 m is normalised to 0.5 and 600 m to 2.0.](image-url)
ASSESSING THE POTENTIAL OF LTCC MINING FROM EXPLORATION DATA

The parametric study suggests that the depth of mining, coal strength and top coal thickness are significantly more important in determining the caveability of top coal than the other parameters examined. These three parameters are usually available from exploration drilling and hence the CSIRO ACARP study offers the possibility of assessing the suitability of a seam for LTCC from exploration data. Multi variant linear regression of the parametric study on these parameters alone gives the formula:

\[
CI = -0.0068 + 5.02e-5 \times H - 7.00e-4 \times CS + 5.25e-4 \times TC - 6.53e-5 \times IR - 1.74e-6 \times CC + 4.93e-6 \times MR \tag{2}
\]

Where CI is henceforth called the Caving Index

As a guide to the relative importance of these parameters on the CI the standard regression coefficient of each parameter is calculated as:

\[
\begin{align*}
H &= 3.08 \\
CS &= 0.54 \\
TC &= 0.47 \\
CC &= 0.22 \\
IR &= 0.15 \\
MR &= 0.008
\end{align*}
\]

Where:

- \(H\) = mining depth (m)
- \(CS\) = uniaxial coal strength (MPa)
- \(TC\) = top coal thickness (m)
- \(CC\) = chock capacity (tonnes)
- \(IR\) = immediate roof strength (MPa)
- \(MR\) = main roof strength (MPa).

Taking now only the three most important parameters based on the size of their regression coefficients, a multivariate regression was undertaken to develop a new simplified equation. Figure 7 shows a plot of the three parameters.

**Figure 7 - Three most important parameters as determined from the standard regression coefficients**

Some selected results from the parametric study described previously are presented below in Figure 8 to Figure 10. In each case the parameter being varied is on the horizontal axis and the dependent variable on the vertical axis. The dependent variable in each case is an average measure of the horizontal plastic strain (non recoverable damage or fracturing) in the top coal immediately above the chock.
Figure 8 - Plastic strain verse coal strength

Figure 8 shows a good correlation between plastic strain and coal strength ($R^2=0.98$) with plastic strain decreasing with a corresponding increase in coal strength. A total change in strain of 0.005 is seen over the test range.

Figure 9 - Plastic strain verse mining depth

Figure 9 shows a good correlation ($R^2=0.98$) between plastic strain and mining depth with plastic strain increasing with depth. The total change in strain of 0.02 was observed over the mining depth range.

Figure 10 - Plastic strain verse top coal thickness
Figure 10 shows a good correlation ($R^2=0.94$) between plastic strain and top coal thickness with plastic strain increasing with increasing top coal thickness. A total change in strain of around 0.005 was observed over the test range.

By considering these 3 most important variables based on the size of the regression coefficients and multiplying the equation by a factor of 1000 (to provide simpler and more meaningful index value) the relationship derived by CSIRO between these variables and CI is then expressed as:

$$CI = -2.64 + 0.0395 \times H - 0.72 \times CS + 0.191 \times TC$$  \[3\]

Chinese experience in LTCC mining provides the possibility to quantify the potential success of mining a seam when the mining depth, coal strength and top coal thickness are known. A database of Chinese mines where the recovery ratio and other required inputs are known has been made available and is used to formulate a simplified relationship between CI and “percentage top coal recovery” by substituting known Chinese LTCC mining statistics into equation 3 and plotting the resulting CI against known top coal recoveries.

Equation 3 relies on easily obtainable data to perform a caveability assessment. Implicit in equation 3 is the possibility of a negative CI as the parametric study was designed around generic Australian conditions and the insertion of Chinese data has produced negative CI results. The reason for this is that the range of the Chinese variables in some cases was outside the range of values considered in the parametric study of Australian conditions. When applying equation 3 based on $H$, $CS$ & $TC$ a negative CI is plausible and it is important to use laboratory coal strength (UCS) for coal strength into equation 3.

A plot of CI verse top coal caving recovery is shown in Figure 11.

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A plot of CI verse top coal caving recovery is shown in Figure 11.

The relationship between CI and percentage recovery of top coal can be determined by creating a line of best fit for the data and is expressed simply as:

$$\text{Percentage of Top Coal Recovery} = 2.72 \times CI + 78$$  \[4\]

### CONCLUSION

A CSIRO - ACARP study of the parameters expected to influence the coal recovery percentages of LTCC has identified depth of mining, coal strength and top coal thickness as the most important factors determining the caveability of top coal. Expressed mathematically they can be summarised in the following equations

$$CI = -2.64 + 0.0395 \times H - 0.72 \times CS + 0.191 \times TC$$  \[5\]

where;

CI is the Caving Index
Chinese experience in LTCC mining provides the possibility to quantify the potential success of mining a seam when the mining depth, coal strength and top coal thickness are known. A database of mines where the recovery ratio and other required inputs are known has been made available and is used to formulate a simplified relationship between CI and “percentage top coal recovery” expressed as:

\[ \text{Percentage top coal recovery} = 2.72 \times \text{CI} + 78.0 \]

The study suggests these parameters influence the magnitude and distribution of the front stress abutment which in turn determines the damage and fracturing to the top coal by exceeding the coal strength, initiating new fractures, and opening existing coal weaknesses on bedding and cleat.

Damage in the top coal is directly related to the caving result, hence the ‘success’ of LTCC mining may be largely determined from this data set (depth, coal strength and top coal thickness) commonly acquired during the appraisal of a mining lease. The use of Chinese historical mining data of LTCC faces allows for these data sets to be related to a top coal recovery percentage figure applicable to Australian mining conditions.

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Zhongming, J. (2006), Theory and technology of top coal caving mining (translation)
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THE LIMITATIONS OF THE OBSERVATIONAL METHOD AND MONITORING PROGRAMS FOR HIGH PRODUCTION LONGWALLS AND AN ALTERNATIVE FRAMEWORK

Ross Seedsman

ABSTRACT: Monitoring is an essential component of the observational method but it is not a substitute for geotechnical design. Roof extensometry is used extensively for managing ground control. It provides an additional and essential level of control for the management of safety but it should not be relied on to provide the necessary warning of interruptions to longwall extraction. Longwall extraction does not have the flexibility to allow for the modifications that are an essential part of the observational method. A more conservative initial design for ground control is required. A logical framework for such design is presented.

INTRODUCTION

Like other branches of engineering, the design objectives in underground mining are structures that are safe, serviceable and affordable. The serviceability criterion applies to the underground roadways themselves as well as to the overall stability of the mine and in recent times to the surface. For mining, the affordability criterion differs substantially from civil engineering in that the economic wealth is produced during the works and not subsequently during the use of the infrastructure. Whilst this introduces some flexibility in terms of the precise location of the excavations during development, it does add requirements with respect to continuity of extraction. This need for regular planned production of coal is even greater for the new generations of longwalls that require coal flows in excess of 500 000 tonnes/month.

Rock and coal are complex materials. There is a large degree of uncertainty related to the ability to adequate characterise them in rationally designed engineering geology studies. Furthermore, their behaviour may be controlled by their high compressive strength or, perversely, their lack of tensile strength and very low strength shear strength along joints and bedding. The science of rock mechanics has evolved to study these materials, and finds application in both the civil and mining sectors. The practice of rock engineering deals with the uncertainties presented by the geology of rock and coal and requires a number of different strategies in the design and implementation process.

THE OBSERVATIONAL METHOD

The highly variable nature of rock and coal masses makes prediction of ground conditions at specific locations impossible. The observational method (Peck, 1969) recognises this. The observational method in ground engineering is “a continuous, managed and integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate.” The objective of the observational method is to achieve greater overall economy without compromising safety. It also gives flexibility in the management of contracts. Because of this flexibility, it is often considered to be ideal for mining. In this definition it can be seen that monitoring is just one component.

Key components of the method are:

- It requires prior assessment of the range of likely ground conditions and excavation/support strategies so that the most probable can be chosen for construction
- The construction methodology must be demonstrably robust so that the flexibility is available
- The responses to monitoring are previously defined
- The responses can be implemented in a timely manner.

There are several problems in applying the observational method to longwall mining. Firstly there is no history or tradition in assessing a range of conditions, in fact the effort has been in determining the minimum support. The level of analyses that have been applied is poor and only applied to one presumed geological condition. While the flexibility to respond to monitoring may be present in development mining, it is not present in longwall production which, to sustain the necessary production rates, requires face retreats of 20 m – 30 m/day.

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The observational method, and its component of monitoring is not a substitute for geotechnical design. In fact, it obliges a greater level of design effort than currently conducted so that the range of geological uncertainty can be anticipated and managed.

**DESIGN IN ROCK MECHANICS AND THE STRATA MANAGEMENT PLAN**

In the face of geological uncertainties, rock mechanics design protocols have evolved Bieniawski (1993). Figure 1 shows a design wheel where steps 3-9 cover the technical aspects of excavation design (see later). Steps 10 onwards are well covered in the strata management plan process that is now part of coal mining in Australia. Most of the steps in Figure 1 can be found in recent mining regulations.

![Figure 12 - A rock mechanics design methodology](image)

Monitoring of roof movements is just one part of the whole design process. Monitoring has no value unless thresholds are set. Over the last two decades, thresholds for roof movement in roadways have been proposed and empirically validated. The monitoring results, combined with higher levels of knowledge at the face supervisory level and intrinsically safer method of work have improved workplace safety. In some cases monitoring does not give adequate time to allow a response and in other cases if poorly interpreted can lead to massive oversupport.

Currently site strata management teams (SMT) tend to operate in isolation of the overall excavation design process and there is insufficient feed-back to allow improved design. Figure 2 presents a way in which the strata management at the site can be better implemented into the longwall planning process by creating feedback links to the overall longwall process.

**MONITORING**

Monitoring assists in managing a safe work place. In the absence of detailed consideration of a range of geological conditions, there is a possibility that the interpretation of thresholds is inappropriate, leading to too many false positives, or even false negatives. In addition to magnitudes of movement, movement rates are being used to guide management decisions regarding secondary support. The longwall acceleration position is defined as when the roof movement in a maingate first exceeds 10mm/week (Thomas and Wagner, 2006). Plots of longwall acceleration positions against depth show a distribution that is remarkably well bounded by the Peng and Chiang (1984) relationship for vertical stress abutments.
The presumption is that movements continue to accelerate at a manageable rate once this 10mm/m threshold is exceeded and that there is still time to install secondary support. While accelerating movements are typically of a roof exposed to increasing deviatoric stress (as the rock fails and the supports yield), rapidly accelerating and stick-slip movements can develop in a low stress environment. Consideration of Figure 3 indicates that stick slip movements may also produce movements in excess of 10mm/week and cannot be resolved unless monitoring is conducted at closely spaced intervals. A better understanding of roof deformation mechanism could lead to more appropriate responses.
GEOTECHNICAL ANALYSES

The essential difference in design in rock mechanics lies in steps 4 and 5. There is a need to reduce the complex geology to something tractable and then to deduce likely behaviour. Rock mechanics design requires both inductive and deductive reasoning, together with heuristics and engineering judgement. The predictions will not be perfect which is why steps 10 onwards (Figure 1) and SMTs exist.

The complexity in rock mechanics comes from the need to identify the presence and interaction of discontinuities. Figure 4 recognises five different approaches to the formulation of the geotechnical model and the subsequent analyses. All have strengths and weaknesses, and that is why the recommendation is always to use at least two.

The five approaches are:

- Precedent/practice – if there is confidence that the geology and stresses are the same, continued use of a successful support regime is a legitimate strategy.
- Rock mass classification – numerical values are assigned to parameters considered likely to influence behaviour, these are combined into a rating and this is used to access a database of behaviour.
- Continuum numerical codes – a rating system is used to reduce laboratory scale continuum properties to values for a large-scale mass that behaves as apparent continuum with reduced strength and deformation properties. Analyses can be done in programs such as FLAC or Phase2, with calibration to mine behaviour.
- “Limit equilibrium” – Based on observations of failure and collapse, failure geometries are proposed and an analysis conducted for an equilibrium of driving and restraining forces at the stability/failure limit. The mathematics involved is often relatively simple. Validation to previous mining outcomes is required.
- Blocky numerical codes – maintain the complex discontinuity geometry and analyse behaviour of blocks without presuming the failure mode using codes such as UDEC and 3DEC.

Precedent/practice and classification schemes work well in rock masses and circumstance for which they were originally developed, for example within one mine or a set of closely related mines. However, Brady and Brown (2004) caution..."Although the use of this approach is superficially attractive, it has a number of serious shortcomings and must be used only with extreme care. The classification scheme approach does not always fully evaluate important aspects of the problem, so that if blindly applied without any supporting analysis of the mechanics of the problem, it can lead to disastrous results."
Continuum numerical codes are readily available and have been used extensively in the last decade. They should be used with care at roadway scale as roof behaviour may be dominated by one or two discontinuities. Continuum codes are excellent for determining the stresses around excavations, the problem is the input of failure parameters, in particular the common assumption that the tensile strength of a rock mass is about 1/10 of the compressive strength and not the conventionally accepted assumption that the tensile strength is zero (due to presence of discontinuities). Blocky numerical codes are still computationally intensive and most likely suited only for academic research.

The limit equilibrium method is not well developed. In recent years it has often been considered as unnecessary in the coal sector in the face of sophisticated numerical codes. The author is currently developing an approach for underground coal. Coal mine geometry is much simpler than the typical metal mine or tunnel. The roof of development roadways consists predominantly of rectangular prisms with axes of the openings and the principal stresses being coplanar with the discontinuities - joints and bedding (Figure 5). This simple geometry allows the application of the logical framework of Brady and Brown (2004) which was initially proposed for one or two discontinuities and that they now note is applicable for moderately jointed rock masses.

Failure and subsequent gravity collapse modes for an assemblage of rectangular prisms (Figure 5) include:

- Compressive failure of the rock substance if the lateral stress is compressive and the deviatoric stresses in the roof exceeds the compressive strength.
- Gravity fall of joint blocks if the lateral stress is tensile.
- Delamination/buckling along bedding partings under self weight and imposed compressive lateral stress.
- Shear along non-vertical joints such that the roof is unstable for all applied stress conditions.

For the longwall application, there is a need to recognise that the roof can undergo a range of stress conditions from development to being left in the tailgate behind the retreat face (Seedsman, 2001). There is also a need to recognise that the stress regime in stone is different from the regime in coal (Seedsman, 2004).

For stone, observations and measurements indicate that high deviatoric stresses in the face/maingate corner may cause compressive/shear failure. In some cases, when very low rock strengths are present, deviatoric stresses may also be high enough in the initial development. Stress conditions in tailgate are more controversial. Seedsman (2001) argues that roof stresses may be tensile due to adjacent goaf and yielding chain pillars; Colwell and Frith (2006) argue that the stresses must be compressive. The only measurements of tailgate stresses are in a coal roof at Ulan (Shen et al, 2006) and these suggest stress reductions. Seedsman (2004) argues that for coal the roof stress on development are very low, suggesting greater stability at the maingate corner and major concerns in the tailgate.
The logical framework (Figure 6) starts with a test for the presence of angled surfaces which provide an intrinsically unstable geometry for all stress assumptions. The next test is for compressive failure which can be readily implemented by comparing the rock strength (either laboratory or sonic derived) with the vertical stress estimated from depth of cover. This vertical stress is a proxy to the deviatoric stress that is acting in a stone roof. From field observations, there is a possibility of the onset of compressive/shear failure concern if this ratio (referred to the roof strength index) is less than 3.5. The next check is for the possibly onset of tensile roof stresses. If the roof stress remain compressive, the support design proceeds based on the hazard of the presence of closely spaced bedding partings.

It is essential to recognise that designs in rock mechanics are predictions on which to base subsequent decisions and the formulation of risk management strategies. In the context of soils engineering, Lambe (1973) discussed how designs/predictions are limited by both the method used and the data available and that a balance is required (Figure 8) to maximise the accuracy of the prediction. It is considered that this observation certainly applies to rock engineering in 2008 (Figure 7).

**CONCLUSIONS**

Observation and monitoring are essential component to any engineering venture to demonstrate performance. But monitoring is not sufficient to assure performance. Acceptable performance comes from integrating monitoring into a geotechnical design process that recognizes the limitation of the observational method for retreating longwalls. The difficulty to adequately characterize the inputs necessary for analysis is not an excuse for failing to commit to improve the economic performance and reliability of the longwall method.

**ACKNOWLEDGEMENTS**

This paper is part of the ACARP funded project CL4029: “Anticipation of geotechnical hazards and planning the response –a tool box of techniques for roadway driveage” being conducted by the University of Wollongong and Seedsman Geotechnics Pty Ltd.
ARE THERE SURFACES DIPPING AT LESS THAN 70 DEGREES AND PARALLEL TO THE ROADWAY?

Y

REDESIGN LAYOUT SPECIAL WORK PRACTICES

N

IS THE ROCK STRENGTH/VERTICAL STRESS RATIO GREATER THAN THRESHOLD (DEVELOPMENT OR MAINGATE)

Y

ARE THE ROOF STRESS VERY LOW? (POSSIBLY COAL ROOF OR IN THE TAILGATE?)

N

DELAMINATION BY SHEAR ALONG BEDDING

N

GRAVITY COLLAPSE OF BLOCKS

Y

COMPRESSION FAILURE OVER-RIDES BOLTS

DENSITY INCREASES WITH DEPTH PATTERN BIASED TOWARDS RIBLINE VERTICAL OR OUTWARD FANNING BOLTS

SUSPENSION FROM ABOVE PILLARS STANDING SUPPORT

SUSPENSION USING LONG TENDONS

Figure 6 - Logical framework for coal mine excavation design
Figure 7 - Design can be limited by the method or the data

REFERENCES


Thomas and Wagner 2006 Maingate roof support design and management during longwall retreat in the Australian coal industry. In 25th International Conference on Ground Control in Mining, Morgantown. pp 191-196.
TAILGATE 802 - GRASSTREE MINE: A CASE STUDY IN PRAGMATIC ROADWAY ROOF SUPPORT DESIGN

Mark Colwell¹, Russell Frith², Guy Reed³

ABSTRACT: In February 2007 Colwell Geotechnical Services was commissioned by Anglo Coal's Grasstree Mine (Grasstree) in the Bowen Basin of Central Queensland to assess the future roadway serviceability and secondary roof support requirements associated with the tailgate of LW 802 (i.e. TG 802). In most instances the ALTS (Analysis of Longwall Tailgate Serviceability) Design Methodology can be directly applied to undertake such an assessment. Whilst ALTS formed the basis for the secondary roof support strategy for the vast bulk of the tailgate, there were two particular aspects associated with TG 802 that required the use other design techniques both in combination with and in addition to ALTS.

Firstly, the gateroad development associated with Grasstree is based on a 3-heading rather than the typical 2-heading configuration employed by Australian Collieries (upon which the ALTS database was formulated). To cater for this in assessing tailgate serviceability, ALTS was combined with its US counterpart ALPS (Analysis of Pillar Stability). Secondly, the installation face of LW 802 was located approximately 260 m inbye of the start of LW 801 and due to the tailgate's orientation and direction of longwall retreat in relation to the major horizontal stress direction, a significant stress concentration acting across the tailgate roof was predicted as the face of LW 802 approached and passed the installation face of LW 801. This situation is sometimes referred to as a “Super Stress Notch”. An analytical approach was adopted when assessing the secondary roof support requirements associated with this section of TG 802.

While this paper summarises the process by which the secondary roof support strategy was developed and subsequently implemented for TG 802, the paper primarily focuses on three issues 1) characterisation of the roof, 2) the analytical design procedure associated with the “Super Stress Notch” zone and 3) what roadway performance outcome constitutes a successful design.

INTRODUCTION

Grasstree Mine (Grasstree) is located in the Bowen Basin Coalfield of Central Queensland and is operated by Anglo Coal (Capcoal Management) Pty. Ltd. The resource area is traversed by the Grasstree Dyke, a 10 m to 15 m thick very strong dolerite dyke that effectively divides the reserves into two separate mining blocks, referred to as the 900’s block on the north side and the 800’s block on the south side.

In relation to Grasstree, mining in the German Creek Seam commenced in November 2003 with gateroad development initially focused on the 800 series longwall panels. Longwall extraction commenced in Longwall Panel 801 (LW 801) in September 2006 and currently extraction is taking place in LW 802. Figure 1 depicts the location of the 800 and (proposed) 900 series longwall panels with respect to the adjoining mining areas of Central, Southern and Bundoora Collieries. The 800’s are bounded to the west by the Cattle Yard Fault and to the east by another system of major faults. The northern limit is the Grasstree Dyke, whilst the southern limit is determined by the lease boundary.

Prior to the extraction of LW 802, Colwell Geotechnical Services (CGS) was commissioned by Grasstree to assess the future roadway serviceability associated with the tailgate of LW 802 (TG 802, which is also designated as ‘A’ Heading - MG 801 in Figure 2) and if necessary recommend secondary support strategies so as to maintain an adequate level of tailgate serviceability during the extraction of LW 802.

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Figure 1 - Site location
Figure 2 – Depth of cover
PREVIOUS MINESITE EXPERIENCE

Except in proximity to significant geological structure (i.e. faulting) the roof of all roadways was simply supported with the primary support installed off the continuous miner, which incorporated 6 x 1.8 m X-grade bolts every 1.3 m with an applied pre-load of approximately 8 t while utilising full roof mesh. The 4:2 staggered pattern employed is essentially a row of 4 bolts alternating with a 2 bolt row, where the row spacing between the alternating rows is 0.65 m resulting in 6 bolts every 1.3 m.

A substantial proportion of LW 801 had been extracted when commissioning CGS to undertake this study and it was found (based on observation and Tell-Tate data) that during the extraction of LW 801, all associated roadways (i.e. belt road, travel road & tailgate) exhibited adequate roof stability (i.e. minimal roof movement & few stability concerns) without the use of any secondary roof support (except in proximity to significant geological structures i.e. faulting or for operational purposes at the “mouths” of cut-throughs). Therefore the 6 bolt every 1.3 m primary support pattern provided satisfactory roof reinforcement and the use of roof mesh adequately contained any surface slabbing.

SELECTING THE APPROPRIATE DESIGN TECHNIQUE

When assessing tailgate serviceability and roof support requirements, generally the ALTS (Analysis of Longwall Tailgate Serviceability - Colwell et al, 2003) Design Methodology can be directly applied to undertake such an assessment for an Australian Colliery. However (as illustrated in Figure 2) in this instance the maingate development associated with Grasstree is based on a 3-heading rather than the typical 2-heading configuration employed by Australian Collieries (upon which the ALTS database was formulated).

ALTS was developed using an Australian database and is specific to 2-heading gateroad development due to the fact that this is by far the dominant gateroad development configuration utilised by Australian longwall operations. However, ALPS (Analysis of Pillar Stability - Mark et al, 1994) was developed based on US data and is specific to 3-heading gateroad development.

Both techniques are utilised to assess tailgate serviceability and the corresponding chain pillar width(s) and ground support required to maintain satisfactory roadway conditions throughout the longwall extraction cycle. A straightforward method of combining both techniques to assess the 3-heading gateroad configuration for Australian conditions is described below.

In relation to this study ALPS was only utilised to assist with the chain pillar evaluation as any roof support recommendations associated with ALPS should not be directly applied to Australian collieries.

Within ALPS the parameter utilised to quantify the chain pillar systems’ contribution to overall tailgate serviceability is referred to as the ALPS Stability Factor or ALPS SF and within ALTS it is referred as the Tailgate Stability Factor or TG SF. Both values are essentially calculated in an identical manner utilising the abutment angle model to calculate pillar load(s) and the Bieniawski (1992) pillar strength equation to evaluate the “strength” or load bearing capacity of the pillar system. The ALPS SF & TG SF are not Factors of Safety; they are pillar ratings employed in the respective analyses and design procedures.

The recommended chain pillar factors for ALPS and ALTS are respectively referred to as the ALP SF_R and TG SF_R, and both parameters are directly related to the Coal Mine Roof Rating (CMRR, refer Mark & Molinda, 2003) via the following equations:

\[
\text{ALPS SF}_R = 1.76 - 0.014 \text{ CMRR} \quad (1)
\]

\[
\text{TG SF}_R = 2.881 - 0.0343 \text{ CMRR} \quad (2)
\]

To convert an ALPS SF for a 3-heading gateroad pillar system to an equivalent TG SF (for the CMRR under consideration), which can then be utilised within ALTS to firstly assess the suitability of the chain pillar sizing in terms of the Australian database and secondly the associated roof support requirements, the following equation is employed:

\[
\text{Equivalent TG SF} = \text{Calculated ALPS SF} \times \left( \frac{\text{TG SF}_R}{\text{ALPS SF}_R} \right) \quad (3)
\]

Once an Equivalent TG SF was established, the ALTS technique could then be directly applied in the evaluation of secondary roof support requirements to maintain satisfactory tailgate serviceability of TG 802 outbye of 22 C/T.
Secondary Roof Support Evaluation and Design – inbye of 22 C/T

ALTS specifically relates to the serviceability design of tailgates subject to double pass longwall extraction. When acting as the travel road, the roof is typically subject to some level of in situ horizontal stress concentration due to longwall retreat. This is typically referred to as Maingate Stress Notching and in terms of the travel road the level of horizontal stress increase will depend on several factors, including chain pillar width (i.e. separation of the travel road from the retreating longwall face), direction of longwall retreat and orientation and intensity of both the major and minor horizontal stress.

As the longwall face retreats, the adjacent goaf prevents any further concentration of the in situ horizontal stress within the roof of the travel road when sufficiently inbye of the faceline or when acting as the tailgate of the next longwall panel. In this instance ALTS provides the means by which a compromise (or the interaction) between chain pillar width and roof support can be assessed in terms of satisfactory gateroad performance.

However, that section of TG 802 inbye of 23 C/T (Figure 2) is subject to single pass longwall extraction and without the adjacent goaf of LW 801 it is considered highly likely that this section of TG 802 will be subject to horizontal stress notching effects during LW 802 retreat.

Down-hole stress measurements indicate a fairly consistent north-northeast direction with respect to the major horizontal stress. The gateroads are orientated at approximately 166° Grid North and therefore in relation to TG 801 and that section of TG 802 inbye of 23 C/T (Figure 2), it was found (based on the down-hole stress measurements) that the major horizontal stress is orientated at an angle in the order of 25° to 45° to the gateroad direction. Such an orientation would likely result in a concentration of the major horizontal stress acting across the “unprotected” tailgate roof as a result of longwall retreat.

Furthermore the area adjacent to 23 C/T will be subjected to a double notch (i.e. concentration) of the major horizontal stress as LW 802 approaches and passes the installation roadway associated with LW 801. This double notch (or concentration) of the major horizontal stress is sometimes referred to as a “super stress notch” or “super-stressing” of the tailgate. The recommendations for this Super Stress Notch zone extended from 25 C/T to 22 C/T to allow for both the initial onset of the additional stress increase and any possible horizontal stress increase extending outbye of 23 C/T.

While ALTS can be adapted to assess roof support requirements for tailgates not protected by an adjacent goaf (e.g. TG 801 in Figure 2), it was decided that in this instance an analytical approach (as described by Frith & Colwell, 2006) could best be utilised to back-analyse/compare the behaviour of TG 801 to assist in designing the secondary roof support strategies associated with the Super Stress Notch zone anticipated for TG 802.

COAL MINE ROOF RATING (CMRR)

The critical input parameter utilised by both ALPS and ALTS for the assessment of tailgate serviceability and roof support requirements/design is the CMRR. When calculating the CMRR an important component is a rock unit’s fracture spacing, which also happens to be one of the critical input parameters associated with the analytical model utilised to assess the secondary roof support requirements associated with the Super Stress Notch zone of TG 802.

Grasstree were able to provide approximately 20 boreholes (near or adjacent to TG 802) of which 17 were suitably geotechnically logged to ascertain credible CMRR values. Those 17 borehole locations are detailed on Figures 2 and 3.

Ward (2006) reports the immediate roof (primarily in terms of longwall geomechanics) is traditionally taken as the strata overlying the German Creek Seam up to the Corvus 2 Seam (approximately 18m thick). This section of roof is customarily divided into five separate geomechanical units (ROF1 to ROF5 in the Capcoal database). However the CMRR is specific to the primary bolt length and is utilised within ALTS to assess roadway roof performance and associated roof support requirements. For this study it is only roof units ROF1 and ROF2 that affect the CMRR.

ROF1 is used to identify the first layer of the immediate roadway roof when it is significantly weaker than the overlying stratum. It is for the most part a fine grained laminated micaceous sandstone, interlaminated to varying degrees with siltstone. In the 700’s it frequently contained fossilised ripple marks on some bedding planes, which tended to encourage delamination. This has also been identified in the 800’s.

ROF1 has been interpreted to have an average sonic derived UCS strength of about 50 MPa, but drops to as low as 20 MPa in a few isolated cases. The laboratory testing associated with the 17 boreholes used in this study returned an average UCS of 36.4 MPa for ROF1 with a standard deviation of 7.5 MPa. In terms of moisture sensitivity, ROF1 is typically classified as not sensitive while occasionally deemed as slightly sensitive. Such an interpretation is consistent with the limited degree of ‘roof flaking’ observed during the underground inspection. Figure 3 shows the thickness contours for ROF1 associated with TG 802 where it typically ranges between 0.7 m and 1.4 m thick.
ROF2 is typically a strong fine to medium grained sandstone, thinly bedded to massive, with micaceous siltstone bands and its (sonic derived) UCS strength generally ranges between 85 and 95 MPa, but it can reach 100 MPa or more. Along TG 802 it ranges between approximately 7 m and 9 m thick. Although ROF2 is thick and strong, it frequently contains one or more thin siltstone bands, usually with some degree of bedding plane shearing that can form potential separation planes.
Calculating the CMRR from borehole core requires that the hole is geotechnically logged (in the splits prior to its placement in the core tray). Specifically, values for fracture spacing, RQD (Deere & Miller, 1966) and/or diametral point load strength ($I_{50}$) are necessary for each geotechnical unit within (and immediately above) the bolted horizon. The fracture spacing (FS) is defined as the average spacing (mm) of actual core breaks or fractures within the geotechnical unit (e.g. if 8 pieces are identified in a 1 m section of core then the FS associated with that 1 m length of core is 125 mm).

It should be noted the CMRR methodology dictates that in terms of the RQD, FS and Diametral $I_{50}$, the value utilised within the individual Unit Rating (UR) calculation is that which results in the lowest UR, however when the FS > 1.22 m then the Diametral $I_{50}$ value should be used.

The boreholes provided by Grasstree were generally suitably geotechnically logged detailing the location and nature of the core breaks, which allowed for a reasonably accurate determination of the fracture spacing associated with roof units ROF1 and ROF2 for most of the boreholes provided. A limited amount of diametral point load strength testing had been undertaken and that was predominantly in relation to ROF1.

It became apparent that utilising the FS in relation to the weaker and often laminaed ROF1 for those boreholes where Diametral $I_{50}$ values were not available would result in an overestimate of ROF1’s Unit Rating and therefore an overestimate of the borehole’s CMRR value. When there are a limited number of boreholes or in this instance where a full suite of geotechnical testing is not available for each borehole or each unit, then to gain a better appreciation of the CMRR (particularly its variability) to utilise for design purposes, it is often valuable to “pool” all of the available borehole geotechnical data in terms of mean and related standard deviation values.

Based on the borehole data, the mean and standard deviation associated ROF1’s Diametral $I_{50}$ and UCS is respectively 0.43 MPa /0.28 MPa and 36.4 MPa /7.5 MPa, while the mean and standard deviation associated ROF2’s Fracture Spacing are 349mm/88mm. Table 1 summarises CMRR calculations for various combinations of these parameters utilising the mean and the mean less one standard deviation with respect to ROF1’s Diametral $I_{50}$ and ROF2’s Fracture Spacing.

<table>
<thead>
<tr>
<th>Case</th>
<th>ROF1 Thickness</th>
<th>Unit No.</th>
<th>Description of Geotechnical Units which form the Immediate Roof</th>
<th>UCS (MPa)</th>
<th>Unit Rating</th>
<th>SBADJ</th>
<th>CMRR</th>
<th>CMRR - SBADJ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.7m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.43$ MPa</td>
<td>36.4</td>
<td>41.7</td>
<td>3.0</td>
<td>54.9</td>
<td>51.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>ROF2 with 349mm fracture spacing</td>
<td>80.0</td>
<td>58.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.43$ MPa</td>
<td>36.4</td>
<td>41.7</td>
<td>4.7</td>
<td>53.8</td>
<td>49.1</td>
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<tr>
<td></td>
<td></td>
<td>2</td>
<td>ROF2 with 349mm fracture spacing</td>
<td>80.0</td>
<td>58.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.3m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.43$ MPa</td>
<td>36.4</td>
<td>41.7</td>
<td>6.0</td>
<td>52.3</td>
<td>46.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>ROF2 with 349mm fracture spacing</td>
<td>80.0</td>
<td>58.3</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>4</td>
<td>0.7m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>4.3</td>
<td>54.6</td>
<td>50.3</td>
</tr>
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<td></td>
<td></td>
<td>2</td>
<td>ROF2 with 349mm fracture spacing</td>
<td>80.0</td>
<td>58.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>6.4</td>
<td>53.3</td>
<td>46.9</td>
</tr>
<tr>
<td></td>
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<td>2</td>
<td>ROF2 with 349mm fracture spacing</td>
<td>80.0</td>
<td>58.3</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>6</td>
<td>1.3m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>7.9</td>
<td>51.3</td>
<td>43.4</td>
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<td>ROF2 with 349mm fracture spacing</td>
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<tr>
<td>7</td>
<td>0.7m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>3.8</td>
<td>53.1</td>
<td>49.3</td>
</tr>
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<td></td>
<td></td>
<td>2</td>
<td>ROF2 with 261mm fracture spacing</td>
<td>80.0</td>
<td>56.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>5.7</td>
<td>51.8</td>
<td>46.1</td>
</tr>
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<td>ROF2 with 261mm fracture spacing</td>
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<td></td>
</tr>
<tr>
<td>9</td>
<td>1.3m</td>
<td>1</td>
<td>ROF1 with $I_{50} = 0.15$ MPa</td>
<td>36.4</td>
<td>37.7</td>
<td>7.1</td>
<td>50.1</td>
<td>43.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
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<td>80.0</td>
<td>56.7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A variation in a unit’s UCS of say 10 MPa does not have a dramatic effect on the eventual CMRR and therefore the UCS for both units is held constant using the average UCS of 36.4 MPa for ROF1 and a realistic minimum value of 80 MPa for ROF2 (Table 1).
Table 1 also summarises the Strong Bed Adjustment (SBADJ) component of the CMRR. One of the most important concepts incorporated into the CMRR is that of the SBADJ. Many years of experience with roof bolting has found that the overall structural competence of bolted roof is very often determined by the quality of the most competent bed within the bolted interval. However current research strongly suggests that primary support densities should be assessed or based on the CMRR minus the SBADJ (last column in Table 1).

To provide an appreciation of the effect of ROF1’s unit thickness on the SBADJ and CMRR, the CMRR calculation is undertaken for three unit thicknesses being 0.7 m, 1 m & 1.3 m. The colour coding associated with Table 1 has the following meaning:

1. Yellow (Cases 1 to 3) indicates that the mean Diametral \( I_{0.50} \) for ROF1 (0.43 MPa) and mean FS (349mm) for ROF2 are utilised to calculate the respective Unit Ratings.

2. Green (Cases 4 to 6) indicates that the mean less one standard deviation Diametral \( I_{0.50} \) for ROF1 (0.43 – 0.28 = 0.15 MPa) and mean FS for ROF2 (349mm) are utilised to calculate the respective Unit Ratings.

3. Blue (Cases 7 to 9) indicates that the mean less one standard deviation Diametral \( I_{0.50} \) for ROF1 (0.43 – 0.28 = 0.15 MPa) and mean less one standard deviation FS for ROF2 (349 – 88 = 261mm) are utilised to calculate the respective Unit Ratings.

Except for a small zone adjacent to 8 C/T TG 802 (refer Figure 3), the thickness of ROF1 is \( \leq 1.3 \) m along the length of TG 802. The CMRR values associated with Table 1 and the CMRR values associated with the individual boreholes strongly suggested that as long as the primary support pattern (of 6 x 1.8 m X-grade bolts at 1.3 m) attained “solid anchorage” within ROF2 and reinforced ROF1 to build a largely self-supporting unit, then a CMRR of between 50 to 55 could be confidently used to evaluate the suitability of the chain pillar design associated with TG 802 (i.e. MG 801) and secondary support requirements.

However the CMRR analyses also contained a strong warning particularly in relation to those zones where the thickness of ROF1 is \( \geq 1 \) m; that is if for any reason “solid anchorage” is not attained by the 1.8m bolt within ROF2 or the primary support density is insufficient to satisfactorily reinforce ROF1 then an effective CMRR of approximately 43 to 46 would result. The recommended support strategy took the above into consideration.

It was decided to characterise TG 802 roof in terms of three zones when utilising ALTS to assess secondary support requirements (outbye of 22 C/T) prior to longwall retreat, with those three zones being:

Zone 1. Where ROF1 \( \leq 0.7 \) m the CMRR is taken to be 54 with a SBADJ of 4. With respect to Figure 3 this applies to those sections of TG 802 from 10½ C/T to 16½ C/T and inbye of 24 C/T.

Zone 2. Where 0.7 m < ROF1 \( \leq 1 \) m the CMRR is taken to be 52 with a SBADJ of 5. With respect to Figure 3 this applies to those sections of TG 802 from 9½ C/T to 10½ C/T, 16½ C/T to just outbye of 18 C/T and 20 C/T to 24 C/T (inclusive).

Zone 3. Where ROF1 > 1 m the CMRR is taken to be 51 with a SBADJ of 7. With respect to Figure 3 this applies to those sections of TG 802 outbye of 9½ C/T and 18 C/T t to just outbye of 20 C/T.

**ANALYTICAL MODEL (Factor of Safety Approach)**

The stability of many engineering structures can be and indeed is evaluated based on a Factor of Safety (FOS) concept, this being a measure of the load applied to that structure in comparison to its ability to accommodate that load without undergoing yield or failure. This is usually expressed as:

\[
FOS = \frac{\text{load bearing ability}}{\text{applied load}} \quad (4)
\]

This approach is commonly used in coal pillar design worldwide with the UNSW Pillar Design Procedure (Galvin et al, 1999) being one such example. In this case the strength of the coal pillar is given by a specific equation that has been determined empirically, based on an industry database of stable and failed pillar cases, typically under reliably inferred Full Tributary Area loading conditions.

Figure 4 details the secondary support installed within these zones.
The Factor of Safety is essentially a risk based measure of the likelihood of the design being inadequate, acceptable values being related to the likely consequence of the design being inadequate and the associated impacts (business, safety or otherwise).

Whilst it is far less common to do so, there is no obvious technical reason as to why roof stability in mine roadways cannot be evaluated and designed for using a similar concept. The problem has always been in being able to reliably assign magnitudes or quantities to the various components of the equation. However as with the approach taken for coal pillar design (i.e. the use of an empirically derived strength equation rather than one based on first principles), industry or individual mine site experience can potentially be used to "calibrate" various elements of the problem and so allow a specific Factor of Safety approach to be adopted.

It is noted though that it is still critical to have a "cause and effect" understanding of the impact of the various technical parameters, simply that assigning numerical values can be based in part on mining experience rather than purely from first principles. For the problem of roadway roof stability, the general design equation can be rewritten as:

\[
\text{FOS} = \frac{\text{load bearing ability (roof strata + roof support)}}{\text{applied load}} \quad (5)
\]

In this instance the applied load acts horizontally across the roof and is a product of the in situ horizontal stress and concentration thereof as a result of the mining process. Therefore the resolution of equation 5 is across the roof, which necessitates that the load bearing ability roof strata and the load bearing ability roof support is also resolved accordingly.

The authors assess (based on industry research/experience) that for small vertical roof displacements (up to around 50mm and possibly to 100mm), slender beam behaviour or buckling is typically the dominant behavioural mechanism occurring within the immediate coal mine roof measures.

Uncontrolled roof behaviour of this type may then lead to other failure mechanisms occurring and to large scale roof displacements or roof falls. By understanding slender beam behaviour, it allows for the most pragmatic way of evaluating the initial load bearing ability of the strata \( (P_{\text{roof}}) \) and subsequently determining the lateral resistance offered by the roof support \( (P_{\text{support}}) \). In terms of the use and application of the analytical model there are three basic components:

1. Evaluation of horizontal stress acting across the roof within individual roof units (typically at various key points in the mining process).
2. Determination of the material properties (including Modulus, UCS as well “beam” thickness & length) associated with the immediate roof units, which are required both in terms of Point 1 above and in evaluating the load bearing capacity of the strata.
3. Utilising a load-balance approach (which incorporates well established load-bearing characteristics of slender beam behaviour and mechanical advantage) a Factor of Safety (FOS) is calculated (refer equation 5). Engineering judgement needs to be applied in selecting a suitable FOS for design purposes this being a risk-based consideration that is always discussed with mine management as part of finalising design outcomes.

It is noted that in the case of design for longwall retreat purposes, the calculated Factor of Safety has the following general definition:

"Factor of Safety against the onset of a process (i.e. stress driven roof deterioration), that if allowed to sufficiently propagate, could lead to a major roof fall".

It is not a Factor of Safety against a roof fall occurring as (a) the conditions under which a roof fall finally occurs are not well defined and (b) practical mining considerations requires that the roof be maintained as stable as possible during longwall retreat so as to minimise any potential impact on face production. Clearly losses can occur by simply excessive roof convergence trapping equipment or deteriorating visible roof conditions necessitating the installation of additional roof support.

Therefore the consequence of an inadequate design is logically the triggering of the longwall retreat TARP and the installation of additional support. It is not the imminent occurrence of a major roof fall and this always needs to be kept in mind when considering the actual magnitude of an adequate design Factor of Safety.

Based on the analyses it became apparent that in terms of overall satisfactory roof performance, the stability of ROF2 was the critical determinant. It was calculated (and subsequently found) that while ROF2 remained stable the installed level of primary support would adequately reinforce ROF1 with respect to the anticipated stress increase. However, should ROF2 buckle (i.e. become unstable) it was considered highly likely the significant roof softening would occur and that such softening could lead to a major roof fall.
Given the critical nature of ROF2 and due to space constraints understandably associated with a technical paper of this type, the following discussion is focused on the analyses associated with ROF2.

Evaluation of Horizontal Stress acting within ROF2

The general equation for the major horizontal stress acting within a rock unit (refer Nemcik et al, 2005) can be written as:

\[ \sigma_H = \frac{\nu}{1-\nu} \sigma_V + TSF \times E \quad (6) \]

where:

- \( \nu \) = Poisson’s Ratio
- \( \frac{\nu}{(1-\nu)} = K_o \)
- \( E \) = Young’s Modulus (in GPa)
- \( \sigma_V \) = vertical stress acting (where \( \sigma_V \) is approximately equal to 0.025 H, MPa)
- \( TSF \) = empirically derived constant (Tectonic Stress Factor)

Therefore by knowing the depth of cover (H) as well as Young’s Modulus and Poisson’s Ratio of the host material, a credible estimate of the major horizontal stress acting within a specific roof unit can now be made. It is noted that in relation to equation 6 Young’s Modulus is quoted in GPa, while the stress outcome is in MPa.

Utilising the down-hole stress measurement data provided by Grasstree and the process of analysis as outlined by Colwell & Frith (2006) a TSF of approximately 0.55 resulted. In conjunction with laboratory testing, the average Young’s Modulus and Poisson’s Ratio were estimated to be 20.08 GPa and 0.187 respectively and therefore for ROF2 equation 6 can be re-written as:

\[ \sigma_H = 5.75 \times 10^{-3} H + 11.04 \text{ (MPa)} \quad (7) \]

A representative depth of cover (H) suitable for back-analysis in relation to TG 801 is 240 m, while the depth of cover (H) associated with TG 802 inbye of 22 C/T is approximately 225 m. Therefore at these respective cover depths the \( \text{in situ} \) major horizontal stress acting within ROF2 prior to mining is estimated to be 12.42 MPa (TG 801) and 12.33 MPa (Super Stress Notch zone).

In terms of the horizontal stress change in the roof that occurs along a tailgate without an adjacent goaf, it will be assigned based on the research findings of Gale and Matthews (1992) whereby they linked the Stress Concentration Factor (SCF for a single stress notch as a multiple of the \( \text{in situ} \) stress) with the angle between the gateroad driveage direction and that of the major horizontal stress (Figure 5).

As previously discussed the gateroads are orientated at approximately 166° Grid North and therefore in relation to TG 801 and that section of TG 802 inbye of 23 C/T, the major horizontal stress is orientated at an angle ranging from around 025° to 045° (average 34.3°) based on the down-hole stress measurement. Such an orientation would likely result in a concentration of the \( \text{in situ} \) major horizontal stress acting across the roof as a result of longwall retreat. Based on the orientation of the major horizontal stress to the gateroad direction and with reference to Figure 5, a Stress Concentration Factor (SCF) for a single extraction panel of 1.6 up to around 2 could apply.

On the basis that a super-stress notch is essentially two horizontal stress notches coming together, a Stress Concentration Factor of around 4 could be argued as being applicable (i.e. a stress notch of an already notched \( \text{in situ} \) major horizontal stress). However a slightly reduced Stress Concentration Factor of 3.5 will be applied for design purposes, this making some allowance for the presence of the chain pillar between the two goafs and the assumption that this would reduce the overall horizontal stress concentration ahead of the second longwall. It is noted that to the best of the authors’ knowledge, no stress monitoring data has ever been collected to fully quantify this issue.

Evaluating the Load Bearing Capacity of ROF2 (\( P_{\text{root}} \))

As previously discussed, roadway roof behaviour and associated instability is primarily based around the uncontrolled buckling of slender horizontal beams under the action of horizontal stress. Early theoretical models for this simply used the concepts of Euler Buckling. However more recent developments by the authors have included other structural concepts that allow a complete range of possible behaviour to be considered according to beam geometry with Euler Buckling representing a relatively small proportion of the full range.

Behaviour outside the Euler range can be defined by a number of different structural concepts. For the purpose of this model use will be made of what is termed as the Johnson formula (see http://physics.uwstout.edu/statstr/statics/ or Beer, Johnston and DeWolf (2006) for more general information on this topic). Utilising these concepts of beam behaviour under axial load in conjunction with the roof’s material & physical properties an estimate of its load bearing ability (\( P_{\text{root}} \)) can be deduced.
In terms of buckling; beams (or columns) have typically been divided into three general types:

(i) Short Beams
(ii) Intermediate Beams, and
(iii) Long Beams.

A short beam will not fail due to buckling, as the ratio of the beam length to the effective cross sectional area is too small. Rather a short, 'thick' beam, axially loaded, will fail in simple compressive failure; that is when the load/area of the beam exceeds the allowable stress. The critical or allowable stress associated with long beams/columns is governed by equation 8 (Euler Formula).

\[
\sigma_{\text{crit}} = \frac{\pi^2 E}{(12L_{\text{eff}}/d)^2} \]

(8)

Where \(E\) is Young's Modulus, \(L_{\text{eff}}\) is the effective beam length and \(d\) is beam thickness.

The above formula only applies while the material is in the elastic region and therefore the maximum allowable stress is limited by the yield strength \(\sigma_y\) of the material, it being taken to be 70% of the UCS herein.

There are a number of semi-empirical formulas for buckling in beams/columns in the intermediate length (and short) range. One of these is the J.B. Johnson Formula. The J.B. Johnson formula is the equation of a parabola with the following characteristics. For a graph of stress versus slenderness ratio, the parabola has its vertex at the value of the yield stress on the y-axis. Additionally, the parabola is tangent to the Euler curve at a value of the slenderness ratio, such that the corresponding stress is one-half of the yield stress. For further information refer http://physics.uwstout.edu/StatStr/statistics/Columns/cols62.htm.

The Johnson equation for the allowable stress is as follows:

\[
\sigma_{\text{crit}} = \left[1 - \left(L_{\text{eff}}/t \right)^2/\left(2C^2\right)\right] \sigma_y
\]

(9)

Where \(t\) is beam's Radius of Gyration and \(C\) is the beam's Critical Slenderness Ratio

\[t = I/A\] and \(C = \left(2\pi^2E/\sigma_y\right)^{0.5}\)

Where \(I\) is the beam's moment of inertia and equals \(bd^3/12\) and \(A\) is the cross-sectional area of the beam (i.e. \(A = bd\)). Note for plane strain analysis the beam width, \(b\), equals 1 m.

Figure 5 - Relationship between horizontal SCF and angle of gateroad to stress direction (after Gale and Matthews 1992)
Essentially when the beam's Slenderness Ratio ($L_{wef} / h$) is greater than the beam's Critical Slenderness Ratio (C) then equation 8 is used to calculate the beam's load bearing capacity and when the beam's Slenderness Ratio less than C then equation 9 is invoked.

Therefore in undertaking these analyses with respect to ROF2 the information required is Modulus (E) and $\sigma_y$ (where $\sigma_y = 0.7 \times UCS$) of the rock unit and the beam's effective length ($L_{wef}$) and thickness (d). As previously discussed for ROF2, its Modulus (E) is taken to be 20.08 GPa while a realistic minimum value for the UCS is 80 MPa and therefore its yield strength ($\sigma_y$) is taken to be 56 MPa.

In terms of the individual beams that will form within ROF2; firstly it is assumed the end fixing condition is pinned and therefore $L_{wef}$ equals the roadway width of 5.2 m and secondly the beam thickness is equal to the fracture spacing as previously discussed under the CMRR section of this paper.

Therefore based on the above for an ROF2 beam thickness of 349 mm (i.e. average fracture spacing) the Slenderness Ratio equates to 51.6 while C equals 84.1 and therefore the load bearing capacity equals 45.5 MPa. Consistent with the CMRR calculations an estimate of the load bearing capacity was also made for a beam thickness of 261 mm (mean less one standard deviation) and based on this beam geometry a load bearing capacity of 37.2 MPa is returned.

It is understood that during the extraction of LW 801, TG 801 exhibited adequate roof serviceability with few stability concerns (except in proximity to significant geological structures) without the use of any secondary roof support. In relation to the ROF2 unit, the horizontal stress acting at the tailgate corner with the longwall face is taken to be a maximum of 24.86 MPa (i.e. $2 \times 12.43$). When this is compared to the allowable stress range within this unit at 5.2 m width of approximately 37.2 MPa to 45.5 MPa, a Factor of Safety (in terms of stability) at the TG corner of 1.50 to 1.83 is calculated. This outcome is judged to be consistent with the satisfactory extraction experience in TG 801.

The horizontal stress assumed to be acting in the ROF2 unit in the Super Stress Notch zone is some 43.2 MPa (i.e. 3.5 x 12.35 MPa). As previously indicated the allowable stress range for the ROF2 is approximately 37.2 MPa to 45.5 MPa, giving an overall Factor of Safety range without secondary support of 0.86 to 1.05. This was judged to be inadequate given that the load bearing capacity offered by the primary roof support is critically dependent on the bolt or tendon anchoring above the height of softening (i.e. anchoring within a stable ROF2).

Achieving an overall Factor of Safety in the range of 1.50 to 1.83 (as found for ROF2 from the back-analysis of TG 801) required an additional ≈30 MPa of load-bearing capacity to be offered by longer tendon support. Utilising the concept of Mechanical Advantage inherent in a buckling beam (refer Frith, 2000), it was calculated that this could be achieved by installing 3 x 6.1 m High Strength Tendons/Cables at 1.6 m spacing with an applied pre-load of 30 t. The three tendon pattern was one of equal spacing across the roof at 1.3 m, 2.6 m and 3.9 m in from either rib side and it was recommended that they be post-grouted.

The above design option (for the Super Stress Notch zone) was provided to Grasstree on the basis of a “cribless tailgate”: i.e. such that standing secondary support should not be required and therefore trigger levels should not be exceeded. However (as requested by the mine) several secondary roof support strategies (utilising various hardware including tendon & standing support) were considered. These various strategies included both the incorporation of standing support as well as the exclusion of such roof support (i.e. “cribless tailgate”).

Various options were presented to Grasstree, which were then carefully considered as part of the colliery’s risk assessment process prior to finalising the secondary roof support strategy to be implemented. In considering these various options and recommendations as a part of the minesite risk assessment process the colliery decided on the support levels delineated in Figure 6.

With respect to the zone where the highest concentration of the in situ horizontal stress was expected (i.e. from approximately 50 m outbye of 23 C/T to 50m inbye of 24 C/T) it was decided by the colliery to install 2 x 6.1 m Bowen Cables at 2 m spacing, which were tensioned to 25 t and subsequently post-grouted. Standing support was also installed which incorporated 1 x 1m² Link-n-Lock at 3.5 m centres.

Figure 7 is a cross-sectional view of the primary and secondary roof support installed within the designated Super Stress Notch zone. However it should be noted that in relation to 23 C/T and 24 C/T, 2 x 4.1 m Superstrands at 2 m spacing with an applied pre-load of 25 t had previously been installed.

The standing support was biased toward the blockside ribline so as to create unequal roof and floor spans. The purpose being; that if any buckling of the roof or floor occurs it is more likely to occur on the pillar side as compared to the blockside thereby protecting the tailgate corner of the longwall face.
The secondary tendon support utilised by the colliery within this zone provided approximately 13 MPa of additional load-bearing capacity resulting in an overall Factor of Safety range of 1.16 to 1.35 with respect to ROF2 stability. The standing support is a passive rather active support and there to “catch” the roof should it buckle and displace to level where a roof fall may occur (if the standing support had not been installed). Therefore it does not actively prevent ROF2 from buckling and as such is not included in the FOS calculation.

**RESULTANT TG 802 ROOF BEHAVIOUR**

The roof behaviour associated with Super Stress Notch zone is typified by the Tell Tale data presented in Figure 8. This level of roof movement is considered high by the colliery as evidenced by the colliery trigger levels (also displayed on Figure 8), which are fairly typical by Australian standards as a part of a colliery’s Strata Management Plan. Furthermore it is highly likely that this level of roof movement would have led to roof falls (possibly stopping longwall production) had standing support not also been installed.

In this area the thickness of ROF1 is approximately 0.6 m (Figure 3) and therefore roof softening (i.e. delamination) has occurred for a considerable distance into the ROF2, certainly beyond the primary bolted interval of 1.8 m. As previously indicated, once ROF2 becomes unstable the additional load bearing capacity offered by the primary support to ROF1 would be significantly reduced and in this instance it is highly likely that significant roof softening and displacement of ROF1 would occur. The total roof displacement shown in Figure 8 is indicative of such behaviour.

In deference to the Super Stress Notch zone, the level of roof movement (i.e. centreline deflection) associated with TG 802 outbye of 22 C/T was generally less than 25 mm.

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**Figure 6 – LW802 stress notch support levels**
6.1m Bowen Cables tensioned to 25t

≈ 10°

≈ 10°

1m² LNL

Figure 7 - Roof support installed within the super stress notch zone

LW 802 Tailgate - A Heading CH2110 (40m Inbye of 24c/t)

Level Orange Trigger = 50mm

Level Yellow Trigger = 25mm

Figure 8 - Tell tale data associated with the super stress notch zone
CONCLUSIONS

The slender beam behaviour/buckling model accurately predicted the behaviour of roof unit ROF2 under super stress notch conditions when there was insufficient active reinforcement to resist the increase in horizontal stress. There are few other ground behaviour models (if any) without significant calibration that would predict such behaviour associated with 80 MPa to 100 MPa sandstone.

Although the secondary tendon support pattern (employed within the Super Stress Notch zone) of 2 x 6.1 m Bowen Cables every 2 m resulted in an FOS > 1, significant roof softening and displacement occurred and it is highly likely that roof falls (probably stopping longwall production) would have resulted had standing support not also been installed. It demonstrates that at this stage the resultant FOS needs to be used in a site specific comparative rather than absolute manner, which is typical in geotechnical engineering due to the various uncertainties faced.

The authors’ were advised that (for a number of reasons) the colliery was not in a position to use the preferred secondary tendon support strategy of 3 x 6.1 m High Strength Tendons/Cables at 1.6 m spacing with an applied pre-load of 30t and planned to use the Bowen cables at the reduced density. However it should be noted that the standing secondary levels were not selected on an ad hoc basis and were actually designed utilising the ALTS approach whereby a trade-off between tendon and standing support (within limits) can be assessed in terms of a serviceable tailgate.

Overall the support strategy employed along the full length of TG 802 was successful in terms of preventing any production stopping falls and providing a serviceable tailgate for all other operational considerations without being unnecessarily conservative. The process of site characterisation, back-analysis, design, risk assessment, implementation and monitoring resulted in what would be considered a mining success. However what is a mining success?

The level of roof movement associated with the Super Stress Notch zone strongly suggests that this section of the tailgate was at a moderate risk with respect to a production stopping fall. Given that the analytical model or approach at this stage attempts to balance loads and therefore results in a design for little or no movement (i.e. similar to a slope stability assessment) a challenging question to ask and hopefully resolve is, “what is an acceptable mining FOS as opposed to a civil engineering FOS” (i.e. a colliery roadway as opposed to a highway tunnel).

Case studies such as that from Grasstree demonstrate what can be achieved in geotechnical design using established methods of structural analysis in combination with the diligent collection and use of fundamental geotechnical data. As well as being effective, such methods are transparent in their content and are therefore amenable to audit by third parties, both of which should be mandatory design requirements.

Currently the Analytical Model is being effectively utilised as a consulting tool and when used in this manner is essentially calibrated on a site by site basis. This in fact is typical of how many (if not most) analytical and numerical models are utilised by experienced geotechnical engineers. While the authors’ consider the model has proven itself (on numerous occasions) when utilised in this manner, it is a significant challenge to formulate a process by which the Analytical Model can be effectively utilised industry wide by mine/site Strata Control/Geotechnical Engineers. This is a primary goal of a current industry sponsored project.

REFERENCES


CMRR – PRACTICAL LIMITATIONS AND SOLUTIONS

Justine Calleja¹

ABSTRACT: The Coal Mine Roof Rating (CMRR) is a rock mass classification which was developed empirically, from a database of coal mines in the USA. The CMRR weighs some of the geotechnical factors which may effect the competence of mine roof and combines them into a single rating on a scale from 0 to 100.

The Australian underground coal industry has, in recent years, wholeheartedly embraced this system as a method of geotechnical characterisation. CMRR is a very simple system which is quick and easy for any engineer or geologist to learn and implement. It also provides a standard process and methodology and an output which can be compared between mine sites, for geotechnical characterisation and design which neatly fulfils the current requirements of the Occupational Health and Safety Act and the Coal Mine Health and Safety Regulation 2006.

However, the use of CMRR on its own will potentially lead to flawed geotechnical characterisation and design. The pitfalls of rock mass classification systems have long been known to respected geotechnical experts such as Brady and Brown (1985) who caution, "Although the use of this approach is superficially attractive, it has a number of serious shortcomings and must be used only with extreme care. The classification scheme approach does not always fully evaluate important aspects of a problem, so that if blindly applied without any supporting analysis of the mechanics of the problem, it can lead to disastrous results".

The objective of this paper is to explore the risks and practical limitations associated with the use of CMRR, and to consider strategies and guidelines for the use of CMRR in characterisation and design which will minimise the risks.

OVERVIEW

The CMRR is an extremely valuable geotechnical characterisation tool which can significantly simplify and enhance the identification and communication of different geotechnical regimes; however, the inappropriate or incorrect use of CMRR could potentially lead to severe consequences.

The pitfalls of rock mass classification systems have long been known to respected geotechnical experts:

Bieniawski (1997) stated, "Rock mass classifications on their own should only be used for preliminary, planning purposes and not for final tunnel support".

Hoek and Brown (1980) “recommend classification systems for general use in the preliminary design of underground excavations”.

Brady and Brown (1985) caution, "Although the use of this approach is superficially attractive, it has a number of serious shortcomings and must be used only with extreme care. The classification scheme approach does not always fully evaluate important aspects of a problem, so that if blindly applied without any supporting analysis of the mechanics of the problem, it can lead to disastrous results”.

Karl Terzhagi commented, "I am more and more amazed about the blind optimism with which the younger generation invades this field, without paying attention to the inevitable uncertainties in the data on which their theoretical reasoning is based and without making serious attempts to evaluate the resulting errors."

A person with limited geotechnical expertise could mistakenly believe that they can easily produce a safe, sound geotechnical roof support, mining method or pillar design by calculating CMRR and using it in conjunction with the readily available design tools (NIOSH and Colwell software and other case histories). The risk of this scenario occurring is exacerbated by both the recent changes in coal mining legislation, which has lead to the reduced involvement of the Inspectorate in reviewing geotechnical designs, and the lack of formal requirements and experience for a person to practice as a geotechnical engineer.

Incorrect or inappropriate CMRR results and designs can be calculated as a result of:

- human error, inexperience, or lack of competency;
- variation in data collection and calculation methodology;
- inaccuracies in the input data;

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• limitations in the calculation process;
• limitation of the specific properties included in CMRR;
• limitation of the cases included in the original database;
• limitation of the empirical approach in not being targeted to identify potential failure mechanisms.

Incorrect CMRR results and designs can be calculated as a result of human error, inexperience or incompetency. The implementation of engineering design quality standards in geotechnical design in underground coal is an important measure to reduce the risk of human error. The risks associated with inexperience and incompetency can be mitigated by the implementation of confirmed competency and experience requirements for geotechnical engineers.

The CMRR result can vary by up to 41 points as a result of variability in methodology and random variation in the point load test data. This extreme variability can be reduced by clarifying the methodology (e.g. fracture logging must be done in the splits, only use diametral point load test data if more than 5 tests results are available for the unit). It can also vary by up to 10 points as a result of different observers. This variation alone can mean the difference between indicating a 4-6 bolt support pattern and the potential for extended cuts. As such the CMRR value needs to be considered to be a rough indication of roof strength. It is not a precise measurement suitable for precise design.

Limitations in the calculation method include inadequate de-rating of low UCS roofs and the lack of inclusion of the frequency of weak unfractured planes in the discontinuity rating.

As a geotechnical characterisation tool, CMRR is limited by some of the properties which are not included in the calculation. It is not suggested that these properties should be included in CMRR, but that they do need to be considered in combination with CMRR to facilitate a comprehensive understanding of conditions:
• faults, dykes, igneous structures;
• vertical and sub vertical structures such as joint sets and cleat which are under represented in vertical core;
• rock stiffness (eg. Young’s Modulus);
• triaxial strength (angle of friction);
• pre-existing stress.

It is important to remember that CMRR is a rock mass strength indicator as opposed to a rock mass stability indicator. When using CMRR in determining mine or support design many other factors need to be considered in combination with CMRR to determine design specifications. In addition to the properties above, the CMRR does not take into account:
• mining induced stresses;
• mining geometry such as roadway span or orientation of workings;
• installed support;
• rib or floor conditions.

It is inappropriate to apply empirical systems such as CMRR to situations which lie outside the range of the original dataset. CMRR was developed for the immediate bolted horizon of underground coal mine roofs. As such, it is not likely to give a reasonable rating for shaft walls, or for rib or floor strength. It may not be a good indicator of roof strength for steeply dipping deposits, or for very deep or high stress conditions. Similarly the applicability of the design tools are limited by the databases they were built on. This is clearly illustrated by the differences in design outcomes between the American and Australian design tools for pillars and roof support density.

In many circumstances, CMRR and the associated design tools are not adequately able to indicate the potential for specific failure mechanisms. Roof lithologies which have failure mechanisms largely driven by horizontal discontinuity spacing are likely to have low CMRR results. However the failure mechanisms which are driven by other properties will not necessarily have low CMRR results.

**HUMAN ERROR**

The potential impact of human error in leading to an incorrect CMRR and inappropriate design is significant. The problems with human error are not specific to CMRR, and are relevant to all characterisation and design tools. However, the risk of human error leading to an inappropriate design may be higher than with other approaches because the person doing CMRR does not need to have geotechnical experience or expertise. With most of the other methods of characterisation and design if the person doing the work doesn’t have experience and expertise they will be forced to obtain the assistance of someone who does. A person with experience and expertise is less likely to make a mistake because they understand the reasons for collecting the data and the end purpose as well as the normal range of input data values. They are also more likely to realise that they have made a mistake when they consider the results and compare them to what would be expected.
The risks of human error can be reduced by the proper implementation of a Quality Standard for engineering design. Specifically, the Australian Standard 3905.12/1999 which is the Quality System Guide to ISO 9001 for architectural and engineering design, describes many of the components and practices which should be second nature to any engineering graduate.

Some of the specific elements of quality design which are essential are:

- Performing alternative calculations (4.4.7 Design Verification) – this means doing any calculation which is going to be used for an important purpose, such as roof support design or pillar design twice using a different method to calculate the result. This also means using more than one design approach.
- Comparing the new design with a similar proven design (4.4.7 Design Verification).
- Undertaking tests and demonstrations of the proposed design (4.4.7 Design Verification).
- Having documented procedures to control and verify the design (4.4 Design Control).
- Conducting Internal Audits (4.17 Internal Quality Audits) – carried out by personnel independent of those having direct responsibility for the activity being conducted. It is really important that any design calculations and design process be checked by someone other than the designer to ensure that the designer understands and follows the design procedure correctly. This applies to a wide range of essential skills and competencies, not just design, but also, for example, geotechnical mapping, installation of geotechnical equipment and monitoring and analysis.
- Confirmed competency (4.18 Training) – training is required so that personnel are appropriately qualified and competent to perform the work. This can be demonstrated by formal training followed by the successful performance of work under supervision.

METHODOLOGY

The methods available for determining UCS (unconfined compressive strength) of core include laboratory testing, calculating inferred UCS from point load testing and calculating inferred UCS from the sonic log. Colwell (2003) states that “in practical terms it has been found (for Australian Collieries) that the UCS can be better quantified from the associated laboratory testing and/or correlated sonic logging” than from axial point load testing. This is consistent with the author’s experience. The UCS rating typically makes up around 30% of the CMRR result and variation associated with errors resulting from the use of axial PLT inferred UCS can lead to a difference in the order of 6 CMRR points (e.g. an axial PLT UCS was 7 MPa = UCS rating of 7 vs. a Lab test UCS of 35 MPa = UCS rating of 13, see Figure 7 for another example).

The discontinuity rating is the most heavily weighted factor in the CMRR calculation and can make up around 70% of the final result. Colwell (2003) states that the core should be logged in the "splits" for RQD and Fracture Spacing. Mark and Molinda do not specify when the core should be logged. Depending on the timing of core logging the fracture spacing and RQD can be significantly different. The Figure 1 core photo below shows core which had the fractures marked up in the splits. The fractures which have occurred since are visible in the photo and not marked. It is evident that the fracture spacing in the splits was around 180 mm, but that the fracture spacing at the time of the photo, when the core had been put in the tray, was around 69 mm. This would lead to a difference of 5 CMRR points. In Figure 2 core photo it is apparent that the fracture spacing has reduced from 1000 mm to 140 mm which equates to a difference of 11 CMRR points.

![Figure 1 - Core photograph](image1)

![Figure 2 - Core photograph](image2)

The second major issue is that if fractures are logged in the core trays, rather than in the splits then the CMRR results are likely to be lower than the CMRR results Mark Colwell obtained and used in his database during the development of ALTS and as such a modification factor would need to be applied to be able to use ALTS. Similarly, if the data which Mark and Molinda used to calibrate their core rating and underground exposure rating was based on fractures logged in the core trays rather than in the splits, then a modification factor would be required to be able to use the NIOSH databases and to compare the Australian data with the US CMRR data.
The recommended method for calculating CMRR is to identify distinct geotechnical units and calculate unit ratings for each one. Then the bolt length to be used is entered and the ratings of the units which lie within the bolted horizon are combined and averaged (weighted by thickness). The adjustment factors are then applied for weak contacts, ground water, strong beds and weak overlying strata.

The height of the top unit which is included in the calculation is not required to match the CMRR horizon. As such the top unit may be 1 m thick, but only have the lower 20 cm included in the calculation. The problem with this is that the lower 20 cm of that unit may have slightly different UCS, fracture spacing, RQD, Diametral strength and moisture sensitivity to the full unit.

The definition of a CMRR unit is that it has consistent geotechnical properties, however common sense has to be applied to avoid creating an excessive number of thin units, so it is likely to occur in practice that sections of a unit, if considered separately may vary by as many as 10 CMRR points. The difference between the section of top unit included in the CMRR and the full unit becomes quite important if sensitivity analyses are being done on various bolt lengths. The current method of calculating CMRR (using NIOSH software) makes it difficult to ensure that the height of the top unit is the same as the bolted horizon, unless the bolt length is well established at the site.

Rapid Rating (Calleja, 2006) is a system which allows the unit ratings and CMRR to be easily calculated for any horizon. This involves creating the following tables of data for the full section of core to be considered (6-8m) with values allocated for every possible depth value:

- fracture log data;
- UCS data table (based on sonic inferred UCS and/or Lab tests and/or Axial PLT inferred UCS);
- moisture sensitivity table;
- lithology table;
- diametral PLT data table.

The unit heights and bolted horizon are selected and the values for each of the unit rating inputs can be calculated by averaging the data in each of the tables between the selected unit heights, with the height of the last unit automatically set to be the height of the bolted horizon. Rapid Rating is a program which uses this methodology, but anyone can do it whether they use Rapid Rating, write their own code, or calculate it manually. The additional benefit of using this method, is that it is very easy to go back at a later date and recalculate for other horizons without having to re-log the core.

**RANDOM VARIABILITY OF INPUT DATA**

The discontinuity rating is the most heavily weighted factor in the CMRR calculation and can make up around 70% of the final result. Mark and Molinda (2005) state that the discontinuity rating is the lower of the Diametral PLT Rating or the Discontinuity Spacing Rating (determined from Fracture Spacing and RQD). The NIOSH software includes a table for up to 48 diametral point load test results which can be averaged for any unit. Colwell recommends doing as many diametral PLT tests as you can, whilst maintaining the core length to diameter ratio of more than 1 for each specimen. Unfortunately in practice it is often difficult to get more than 1 or 2 diametral point load tests on a particular unit. This becomes a problem because point load test results (diametral and axial) tend to be highly variable.

In the Figure 3 each value on the x axis represents a Unit number. Each unit represents a different ply within the coal seam. All of the units are coal (some more stoney than others) except for unit 3 which is a claystone. The graph shows all of the Is50 results for 32 holes at a mine site. There are some holes which have 2 or 3 tests on the same ply and the variability for these samples is similar to the overall variability when looking at all of the holes. Specifically, it is evident that the diametral data is almost a uniform distribution (i.e. there is an equal probability of obtaining any value between the minimum and maximum values), rather than a normal distribution where there is a higher probability that any single test value will be closer to the mean than to the minimum and maximum values. At this site, where most CMRR units which are usually individual plies and only have 1 or at best 2 or 3 diametral tests, the discontinuity rating associated with that unit could either be 25 or 41 or anywhere in between despite the fact that the physical properties of that unit are fairly consistent. The graph shows that the variation in Diametral Is50 is effectively random and if you use Is50 in the discontinuity rating and don’t have the luxury of using the mean from a large number of tests e.g. 20 tests, the CMRR result could be randomly variable over a range of 16 points. Alternatively, if based on this data, you use an average Is50 for each ply for all holes then you will end up distinguishing CMRR by the holes which have very low RQD or Fracture Spacing and distinguishing them by variability in UCS. You would be capping the maximum possible CMRR value by the average diametral rating for that ply, which is probably not appropriate in reality, because some holes may have plies with higher average Is50 results (if it were possible to obtain 20 tests from that ply). It is generally not possible to obtain enough test data to
be able to get a reasonable average for an individual ply in a single hole, and as such it may be concluded that it is inappropriate to use diametral Is50 results in the CMRR calculation at this site.

![Diametral pointload test Is50 by unit](image)

The Figure 4 photographs below show diametral results from a different site, where the immediate roof unit is vertically extensive and has consistent properties. A large number of diametral tests were done on this unit and supported the evidence shown in the previous example. Figure 5 shows the Is50 results for this drill hole versus depth of the test. The tests in the graph start at the top of the seam, and are all in stone. The graph shows a general trend of increasing Is50 strength associated with increasing distance above the seam, however at any one point there is a large variation in diametral strengths in the order of 0.5-0.8 MPa, and in two locations the variability is 1.5-2 MPa. For this particular rock type the variability in the rating is typically around 10 CMRR points and as high as 35 CMRR points. This is an example where the rock has had almost the maximum possible number of diametral tests done. It could be argued that in this case a reasonable average for any location could be obtained, although there would have to be some concern about which diametral strength value would be appropriate to use for the locations where the PLT ratings ranged between 25 and 60. It is not very common for so much closely spaced diametral data to be available. At other sites it would be likely that only a quarter of the number of tests would be available which would mean that individual test values would have to be used rather than averages, and this will lead to high variability in the diametral ratings, and as a result in the CMRR ratings which may not be truly reflective of the properties of the units.

**CALCULATION PROCESS**

There are problems inherent in the method of calculation of the Discontinuity Rating. Fracture spacing and RQD are purely a measure of the frequency of discontinuities with less strength than the load applied in the drilling process (which is variable). On the other hand Diametral Point Load testing is purely a measure of the strength of a discontinuity and does not include any spacing/frequency component. As a result, when diametral point load testing is used to calculate the discontinuity rating, the same CMRR value could be obtained for strata with weak bedding planes at 300 mm spacing as strata with weak bedding planes at 50mm spacing.

The presence of weak but unfractured planes in the core is only taken into account by the unit contacts adjustment (unless diametral point load testing is used to calculate discontinuity spacing). The unit contacts adjustment is calculated by determining the number of contacts between CMRR units which are weak. The discontinuity rating can vary from 20 to 60 for a CMRR unit, however the maximum deduction for weak but unfractured planes (from the unit contacts adjustment) is 5.

The UCS of the rock is the second most important factor in CMRR with its rating ranging from 5 to 30 (compared with 20 to 60 for the discontinuity rating) (Figure 6). The author’s experience with the CMRR results from the proportional weightings of UCS and discontinuities seems to give a reasonable result in most circumstances. However, at the lower end of the UCS scale the decrease in UCS rating is linearly proportional to the decrease in UCS from 34.48 MPa down to 0 MPa. Specifically, the difference in UCS rating (and CMRR value) between a 20 MPa and a 5 MPa core is 3. This would not even represent a different CMRR classification. Whilst 5 MPa core is not very common, it is essential to highlight areas where very weak rock occurs (whether it is fractured or not) because substantially different management approaches are required to accommodate the different risks which exist for 5 MPa rock as opposed to 20 MPa rock.
Figure 4 - Core photographs showing diametral PLT Is50 values

Figure 5 - Diametral PLT values and discontinuity ratings versus depth
It can be argued that low UCS rock would end up with a low CMRR as a result of low diametral strength. However this assumption is not 100% reliable, as shown in Figure 7. The laboratory tested UCS was 6 MPa, and this value is consistent with the lab test UCS results from this unit in other holes at the mine site. The photograph shows the Diametral Is50 and axial PLT inferred UCS results for this unit. For some sections of the core the axial PLT UCS results of 9 MPa and 5 MPa were appropriate, but other sections gave highly inaccurate UCS results of 25MPa. Similarly, some sections of the core gave appropriate diametral results of 0.05 and 0.14 but other sections gave results of 0.63 and 0.72. It is interesting to note that the low diametral results occurred with the high UCS results and vice versa, which indicates that the variability in results is not attributable to general variability in the core properties, but rather due to variability in small scale properties of the test specimens. A reasonable CMRR for this unit would be around 30 - 40. If this unit was less vertically extensive and only one set of PLT data was available, e.g. the first or last set then the CMRR obtained purely from point load testing would be 40 - 44 without groundwater. If the more appropriate diametral results of 0.05 and 0.14 were used the CMRR would be 28 without groundwater. This example illustrates that there are significant problems associated with:

- the use of axial PLT data for determining UCS;
- the use of diametral PLT data for determining bedding plane strength;
- the reliance on diametral strength to de-rate low UCS units (the UCS rating should do this not the diametral rating).

LIMITATION OF ROCK PROPERTIES INCLUDED

CMRR is inadequate to be used on its own for the purposes of geotechnical characterisation because of the rock properties which can be very important to rock mass strength and are not included in core calculated CMRR:

- faults, dykes, igneous structures;
- vertical and sub vertical structures such as joint sets and cleat which are under represented, or over represented in vertical core;
- rock stiffness (e.g. Young’s Modulus);
- triaxial strength (angle of friction).
It is possible to calculate CMRR in the vicinity of geological anomalies such as faults, dykes and igneous structures, however CMRR will not take into account potential geometric structural failure mechanisms, or the impact of such structures on the overall mine opening stability (the combined impact of the structure on roof, ribs and floor), or the impact of such structures on in situ and mining induced stress changes. It may be impossible to obtain a “representative” CMRR value in the vicinity of a geological anomaly because of large variations in the geotechnical conditions around the anomaly. In addition, CMRR is an empirical system, and any specific geological anomaly is not likely to be represented within the original dataset. As such, any conclusions which can be drawn about CMRR results based on the original database could not be applied to CMRR values for a specific geological anomaly.

Vertical fractures are typically not the cause of difficult conditions in underground coal mines due to gravitational interlocking or confinement provided by horizontal stress. When vertical fractures occur they are generally under represented in vertical core. However when vertical fractures are closely spaced (e.g. cleat in a coal roof) there may often be large sections of the coal core which is effected by a single vertical fracture and whilst a single vertical fracture will not lead to a very low fracture spacing on its own, it will often lead to many more bedding plane fractures than would otherwise occur, and thus lead to excessively low fracture spacing and an excessively low CMRR.

Sub vertical fractures are similarly under-represented in vertical core. Sub vertical fractures can lead to difficult conditions in underground coal mines, especially if there is more than one joint set. CMRR does not adequately indicate circumstances where stress or gravitational induced block failure can occur, and so where there is a potential for block failure to occur, analytical analysis is necessary to identify the potential failure mechanisms and determine appropriate management strategies.

Rock stiffness (e.g. Young’s Modulus) is not included in the CMRR calculation. Rock stiffness is a measure of the amount of deformation (strain) which will occur under a certain amount of stress (load per unit area). If a roof has rocks which have different stiffnesses, then under uniform strain (a generally accepted condition for underground coal mine strata), the rocks with higher stiffness will be under higher stress and the rocks with lower stiffness will be under less stress.

A common assumption is that Young’s Modulus is usually linearly related to UCS. The implication is that as UCS increases, so does Young’s Modulus and the ratio of UCS to Young’s Modulus remains fairly consistent. This means that although the rocks with higher stiffness would carry higher stress, they would also have higher strength and so would be equally as stable as the lower stiffness, lower stress, lower strength rocks. Therefore the variation in rock stiffness is sufficiently considered by analysing UCS. This is a reasonable argument, but only in situations where all of the strata has a consistent UCS to Young’s Modulus ratio. Unfortunately, in reality there are many situations where this assumption is not true, and identifying those circumstances is essential to characterising strata behaviour and understanding the potential failure mechanisms.

For example, at one site the rock testing database of 205 samples (Figure 8) shows a range in the Young’s Modulus to UCS ratio (x 0.001) between 0.06 and 1.16. Coal measure rocks typically have E/UCS ratios (x 0.001) between 0.2 and 0.3. However the samples in the dataset below demonstrated that some particular units had significantly different ratios. The weak sandstone unit shown in the previous photograph is one example with ratios between 0.4 and 0.7. Sixty three coal samples had ratios between 0.07 and 0.34 with an average of 0.14. The coal sample ratios were markedly lower than the non coal samples which had an average of 0.31.

The lower E/UCS ratio of coal samples has major practical significance. It may provide an explanation for why coal roofs are much more stable than one would expect after considering the typically highly fractured nature of coal and its generally low UCS (and resulting low CMRR values).

It is also very important to identify any units with much higher E/UCS ratios than the surrounding units as these may be the precipitators of progressive stress based roof failure. Units with high E/UCS ratios may be much less competent than their UCS alone would indicate because of the higher levels of stress which they carry. In Australia, where core drilling is standard practice, the collection and laboratory testing of core samples is a small proportion of the exploration cost. There is no reason why UCS cannot be determined from laboratory testing, and the determination of Young’s Modulus is available as a standard component of a UCS test.

Uniaxial Compressive Strength (UCS) is the strength of a sample of rock when it is loaded uniaxially (only in one direction). It is a rough indicator of the strength properties of rock. Triaxial strength provides a more complete picture of the behaviour of rock in situ, as the strength of the rock in one direction is determined for varying conditions of confinement in the directions perpendicular to the primary loading direction. In virtually all circumstances the strength of a rock sample (intact or fractured) will increase associated with increasing confinement, and the rate of increase is consistent, whether the specimen is intact or already fractured. The ratio of increase in strength to increase in confinement is described by the friction angle. Some rocks have very low friction angles and gain very little strength when they are confined. In contrast other rocks have very high friction angles, and may for example have a low UCS but then high strength when under 4 MPa or 8 MPa of confining pressure (e.g. horizontal stress).
For example, a claystone in the immediate roof with a UCS of 45 MPa and a friction angle of 44 degrees would have a strength of 100 MPa at 10 MPa horizontal stress. This unit would actually be stronger than a 59MPa UCS siltstone with a friction angle of 31 degrees which would have a strength of 90 MPa at 10 MPa horizontal stress. It is interesting to note that many of the coal samples in the previous graph which often had low E/UCS ratios also had high friction angles which may also contribute to the higher competency of coal roof.

Triaxial strength testing is more expensive than UCS testing, and for this reason it is not the standard test performed on all core samples, however it is possible to pick a smaller proportion of representative samples out of a testing program and conduct triaxial testing without substantially impacting on the economics of the exploration program.

LIMITATIONS OF OTHER PROPERTIES INCLUDED

It is important to remember that CMRR is a Rock Mass Strength indicator as opposed to a Rock Mass Stability indicator. When using CMRR in determining mine or support design many other factors need to be considered in combination with CMRR to determine design specifications. The CMRR does not take into account:

- pre-existing or mining induced stresses (e.g. resulting from depth of cover, horizontal stress, longwall abutment and stress concentration, stress direction, faults);
- mining geometry such as roadway span or orientation of workings;
- installed support;
- rib or floor conditions.

The PSUP vs. CMRR graph (Figure 9) shows that for any CMRR value, the associated PSUP values can virtually range across the full spectrum for each country. This is due to the impact of all of the other factors, listed above, which combine with CMRR to lead to overall roof stability.

LIMITATION OF CASES IN THE DATASET

CMRR is an empirical system. This means that it has been developed based on a specific set of data (in this case a large number of underground coal mines in the USA). As with all empirical systems, it is generally useful and valid whilst used within the boundaries of the data from which it was developed, but it cannot be assumed to be applicable outside that dataset. For example, the dataset was developed based on mine roadways and would not be directly applicable to a drift which is oriented at an angle to bedding and would potentially have additional wedge failure risks. Following the same logic it may not be a good indicator of shaft wall properties. It is not necessarily a good indicator of floor conditions as poor floor conditions may be more influenced by slake durability and UCS rather than fracture spacing which is most heavily weighted in CMRR. As described previously, if the original core CMRR data was based on fracture logging in core trays rather than in the splits, then a modification factor would have to be applied to compare the original data with data calculated from fracture logging in the splits. Similarly, if any significant changes are made to the calculation methodology, then the modification factors will need to be applied to data calculated using the previous methodology.
The limitation of cases in the dataset is particularly relevant when considering the use of CMRR design tools. The PSUP vs. CMRR graph (Figure 9) clearly illustrates that the support densities used in the USA, Australia and South Africa are almost mutually exclusive. Whilst some of the difference can be attributed to lower average depth of cover in the USA and South Africa, and possibly other factors such as lower horizontal stress. However, the roof fall rates in South Africa and the USA were also higher than in Australia and it is likely that the lower support densities are directly related to higher roof fall rates, which are tolerated to different levels as a result of cultural differences.

CMRR AND FAILURE MECHANISMS

As an empirical system, CMRR is limited by the fact that it does not take into account different failure mechanisms associated with different geotechnical environments.

CMRR is primarily calculated from horizontal discontinuity spacing or bedding plane strength, UCS and moisture sensitivity. Roof lithologies which have failure mechanisms largely driven by horizontal discontinuity spacing (e.g. high angle shear failure of thinly weakly bedded roof) are likely to have low CMRR results. However the failure mechanisms which are driven by other properties (some in combination with horizontal discontinuities) will not necessarily have low CMRR results:

- block failure – determined by boulders or sub-vertical joints;
- skin slab failure – determined by properties of the first 0.2 m of roof rather than the full bolted horizon;
- overstressing failure – determined by in situ and mining induced stress, E/UCS, and triaxial strength properties;
- de-stressing failure – determined by sub vertical joints or mining induced fractures and mining induced reduction in stress;
- tension failure – determined by vertical or sub vertical joints in coal roof, or for bulking failure: the thickness of coal beam, the thickness of roof stone, the bedding plane properties, E/UCS properties, triaxial strength properties, tensile strength properties, and stress field;
- combined structure and stress failure - determined by in situ and mining induced stress, sub vertical discontinuities, E/UCS and triaxial strength properties.

For example, a 50 MPa massive sandstone roof with occasional weakly bonded boulders or with regular open joints at 5 m spacing, 60 degrees to horizontal and oriented parallel and at right angles to the roadway direction could have a CMRR of 60+ which would incorrectly indicate the potential for extended cuts and a low density support pattern. Alternatively a roof with 30cm of very thinly weakly bedded siltstone with 1.7 m of massive unfractured 80 MPa claystone could have a CMRR of 72 and have significant skin failure problems.

LIMITATIONS OF THE DESIGN TOOLS

There are various design tools and case histories which are available to use with CMRR: ALTSII (Australian Longwall Tailgate Design), ALPS (NIOSH Longwall Pillar Stability), ARBS (Roof Bolt Selection), extended cut stability, longwall mining through open entries and recovery rooms. The empirical design tools have similar limitations to CMRR. They are limited by:
• the use of CMRR (its variability and geotechnical factors not included);
• the range of the cases used to build the tool and;
• important factors effecting design which are not included in the design tool.

Some other factors which could potentially result in an unsafe pillar/tailgate design using ALTSII include the presence of very weak floor, the occurrence of faults or other geological anomalies, or the presence of a weak sliding plane such as a clay band which can prevent pillars from developing confinement. In addition to these factors, ALPS does not include consideration of horizontal stress. Problems with design outcomes due to the difference in the cases used are evident when using the same inputs for both programs. For example, ALTSII recommends a larger pillar width varying from 10 m to 25 m wider than the ALPS results for CMRR 45 and DOC 450 m.

ARBS is the “Analysis of Roof Bolts” program developed by NIOSH to provide roof bolt support design parameters based on CMRR, depth of cover and intersection span. It was developed by statistical analysis of a variety of roof bolt systems at 37 mines in the USA with the effectiveness of the systems determined by comparing the number of roof falls per 3000m driveage. ARBS does not include horizontal stress magnitude or the difference in reinforcement performance resulting from the use of point anchored or fully grouted bolts. The data used in the program did not include the effects of longwall loading, only development conditions. The program is based on USA support practices and there is a significant risk that higher roof fall rates than are generally tolerated in Australia, would occur if ARBS designs were applied in Australian mines. This is supported by the recommendation in the ARBS help file that “The field data also indicated that in very weak roof, it may be difficult to eliminate roof falls using typical U.S. roof bolt patterns. When the CMRR was less than 40 at shallow cover, and less than 45-50 at deeper cover, high roof fall rates could be encountered even with relatively high roof bolt densities. Faced with these conditions, special mining plans or routine supplemental support might have to be considered.”

The extended cut stability and longwall mining through open entries and recovery rooms analysis are not design tools per se, but rather a compilation of case histories. The extended cut data was all taken from USA mines and represented in the form of a graph showing CMRR versus Depth of Cover with the points on the graph separated into “Always Stable”, “Sometimes Stable” and “Never Stable”. As described for the previous design tools, there are many other factors which effect extended cut stability and the risks associated with extended cuts than are included in the graph and in the data. The determination of stability was based on interviewing personnel at the specific mines, rather than a quantitative measure. The cultural safety differences between Australia and the USA may also be present in this dataset and it would be inappropriate for a geotechnical engineer to use this empirical database (on its own) to determine that extended cuts could be implemented in an Australian mine.

Oyler’s paper on longwall mining through open entries and recovery rooms analyses factors (including CMRR) which effect whether these operations can be conducted without severe weighting or roof falls. It is a compilation of 130 case histories from USA, Australia and South Africa. The method of determining success or failure of the cases was more easily quantified and so more objective than the extended cut data. The potential cultural safety differences are less significant in this dataset because it includes Australian data, however there are still many other factors which would effect the stability of a recovery room which could not be included in the data and so the use of this data should be limited to a broad indication of the possibility for mining into a recovery room and any design should not be developed without additional extensive analysis using other appropriate design methods to confirm it.

CONCLUSIONS

The Coal Mine Roof Rating is a very valuable tool for geotechnical characterisation and empirical design, however it needs to be used by competent and experienced geotechnical engineers with careful consideration of its limitations.

Risks associated with human error, inexperience and incompetence occur with all characterisation and design methods but are more likely to occur with CMRR because less expertise and experience is needed for its use. These risks can be managed though the implementation of engineering design quality standards.

Variability in the CMRR results can be reduced by ensuring that fracture logging is done in the splits, diametral point load test results are only used where a large number of tests are conducted on each unit and UCS is calculated from lab test data, correlated sonic logging or high density correlated axial point load testing. Variability of up to 10 points is unavoidable due to the observer differences. Therefore CMRR should not be considered to be a precise value but rather a rough indicator of rock mass strength.

CMRR does not adequately de-rate low UCS lithologies and it does not include numerous important properties which are essential components of a thorough geotechnical characterisation. As such, CMRR should not be used in isolation for mine site geotechnical characterisation. It should be used as one component of a broader assessment.
The empirical design tools which can be used with CMRR are important and useful datasets, however, they are also limited by the range of the cases they were developed from, by important geotechnical properties which are not included and by the inherent variability in the CMRR values which are input. It would be unwise to implement an operational geotechnical design based on a CMRR design tool without considering all of the potential failure mechanisms and without employing alternative appropriate design methods to confirm any design outcomes.

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GEOTECHNICAL DESIGN AT A MINE SITE LEVEL – WE HAVE NO CHOICE

Russell Frith¹ and Mark Colwell²

INTRODUCTION

It is not that long ago that strata control was the domain of the Under Manager. When the authors joined the Australian Underground Coal Industry in the late 1980’s, few if any mines or mining companies employed qualified geotechnical engineers and the determination of roof support rules and coal pillar design for example were done largely within mine management. One only has to look at the list of participants at the various UNSW pillar design and geomechanics courses in the early to mid-1990’s to substantiate the above statement.

Geotechnical engineering in the coal industry was seen in a research and consulting role at that time, with specialist geotechnical engineers being employed by the likes of ACIRL and CSIRO and used by mine sites to advise on the causes of roof control problems and unplanned events after they had occurred. It was out of this environment that the consulting firm Strata Control Technology was formed.

By the mid-1990’s, the formalised Strata Management Plan was beginning to evolve, it being focussed on the use of roof monitoring and mapping combined with the traditional observations of miners in order to improve the reactive ability of mine sites to changing strata conditions. The consulting firm Strata Engineering was at the forefront of developments in this area and still regularly publish today on the use of strata monitoring data for decision-making purposes (Thomas 2006). The current situation is that most mines now employ a person in the defined role of the geotechnical engineer, as formalised strata management is now firmly entrenched as one of the core requirements of underground coal mining in both NSW and QLD.

However despite the substantial improvements in strata management practices at coal mines in the last two decades, major strata control losses are still sustained by longwall mines on an occasional basis with consequent large business losses (especially given the high coal prices and longwall production levels compared to 20 years ago). No amount of roof monitoring data, borescope observations, hazard mapping or local mining experience was presumably able to either predict or prevent these losses being sustained. A critical element of the strata control process must at times be missing, which is the subject of this paper, namely effective geotechnical design, particularly as it relates to the role of the mine site geotechnical engineer.

In considering the role and importance of geotechnical engineering design in coal mining, several basic questions will be addressed:

(i) What are the basic elements of engineering design?
(ii) What does the legislation require in this area?
(iii) Does it offer benefit to operating mine sites and the industry in general?
(iv) Why is the mine site geotechnical engineer so important to the future of the coal industry?
(v) Where is the industry up to and what are the potential areas for further development?

In answering these questions, the authors will address the title of their paper “Geotechnical Design at a Mine Site Level – We Have No Choice” and provide a general response to the question posed by Ross Seedsman and his co-authors several years ago in the paper entitled “Chain Pillar Design – Can We?” (Seedsman et al 2005), which discussed the design limitations and various required outcomes for chain pillars at that time.

WHAT IS ENGINEERING DESIGN?

Geotechnical engineering is an engineering discipline and therefore the general requirements of engineering design logically apply. Some of those required elements are listed as follows:

- It is undertaken by suitably qualified and competent engineers (this will be discussed in more detail later in the paper).
- It utilises engineering parameters (e.g. strength of steel, modulus of concrete, applied loads/stresses etc.) that can either be measured or estimated prior to construction.
- It converts engineered based input parameters into a design output that is linked to some form of risk-based measure (e.g. Factor of Safety, probability of failure etc.).

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• The design is implemented or constructed usually under the direct supervision of the designer or team of designers.
• The design methodology is transparent in its content, is numbers based and is amenable to independent review and audit.

What is not engineering design (as it does not meet the above requirements) are strata control practices that unfortunately still find use in the coal industry and are based on such considerations as:

“the roof looks to be in good condition/has only moved 3 mm prior to longwall retreat, therefore secondary support needs for extraction are minimal”
“we’ll do what we have done before because it has worked in the past”

In fairness to the industry, without the availability of well founded geotechnical design methods that are focussed on coal mine strata control, mining personnel have had little option but to revert to site-specific observational “design”, i.e. essentially rely on previous local experience and in many situations this has proven to be effective. However such methods tend to be unreliable when the geotechnical conditions change but go unnoticed. Only when strata control difficulties become visibly apparent (often during longwall retreat) do such changes in conditions become evident, by which time significant business losses are usually sustained even if effective remedial measures are put in place prior to a major fall of ground.

There is also at least one well demonstrated characteristic of mining geomechanics that limits the effectiveness of the observational design approach, that being the step-change in behaviour/condition.

A good example of step-changes in roadway roof behaviour is found in data published by Gale et al (1992) whereby the roof softens in a series of discrete steps that are driven by ever-increasing roof displacement – see Figure 1. Roof behaviour is clearly not gradational (i.e. roof stability is not lost incrementally with increasing roof displacement) and a roof environment that is visibly stable at say 2 mm of movement may in fact be similarly stable at 10 mm but highly unstable at 20 mm with several metres of associated roof softening. The potential for such step-wise reductions in roof stability are a major obstacle to the reliability of observational type geotechnical design as demonstrably, the visible conditions or magnitude of the measured roof displacement prior to secondary extraction will not always provide a reliable basis predicting any associated roof stability changes.

It is noted that in addition to roadway roof behaviour there are similar step-wise changes in such subject areas as mining subsidence, coal rib stability and overburden weighting.

The contention of the authors is that the only way that the industry can improve its ability to predict strata stability prior to mining (whether development or secondary extraction) and prescribe effective control measures is through the routine use of credible geotechnical design that conforms to all of the basic requirements for engineering design listed previously.
Whilst on the subject of geotechnical design, one other subject area is commented upon. It is realistic to suggest that there is a point of view held by a significant segment of the rock mechanics fraternity that numerical modelling provides a researcher/consultant with a tool to undertake real engineering whereas empirical techniques offer only “simplistic formulae” (Tarrant, 2005). It would be naïve for any researcher, whose objective is to provide an underground coal mining industry with a widely accepted empirical geomechanics model, to be unaware of this point of view.

The first point to make is that empirical methods of design are absolutely not “trial and error” as was suggested by Tarrant (Tarrant 2005) in attempting to justify his adoption of a numerical modelling design approach to tailgate strata support in preference to existing empirical methods.

The authors would also like to note that probably the greatest scientist who ever lived (Sir Isaac Newton) made liberal use of empirical methods (that being the use of observations and data in developing sound reasoning) in his research work that ultimately led to his theory of universal gravitation as contained in what is generally accepted to be the greatest science book ever written, his *Principia of 1687* (Bardi 2006). Furthermore those exact same principles were used by NASA nearly 300 years later in arguably man’s greatest achievement, namely the moon landings of the late 1960’s and early 1970’s.

If the use of empirical methods were good enough for Newton in trying to develop an understanding of the universe and NASA in sending man to the moon, they are certainly good enough for the geotechnical engineering fraternity in trying to establish the fundamental laws of strata mechanics in coal mines.

The authors have in the past and will continue into the future to combine empirical observations and data with sound analytical methods and reasoning in attempting to further our understanding of the response of the geotechnical environment to mining and provide improved methods of analysis and design. To the best of the authors knowledge, this is the only approach that has been able to provide mine site geotechnical engineers with the transparent and useable design and assessment tools that they so desperately need. The current availability of such tools will be discussed in more detail later on.

**WHAT DOES THE LEGISLATION REQUIRE?**

Whilst the relevant legislation in NSW and QLD is worded differently, the intent in relation to strata control practices are essentially the same, as follows:

(i) investigate those factors that influence strata stability  
(ii) estimate the likely geological/geotechnical conditions to be encountered  
(iii) estimate strata stability in the conditions likely to be encountered  
(iv) prescribe support measures to ensure strata stability

The QLD Regulations even go so far as to use the phrases “strata support methods shall be designed” and “records of numerical calculations used...” Clearly the intent is for a pro-active and engineering based geotechnical design process. Another legal requirement that all practicing geotechnical engineers should be aware of is AS3905:12 (1999) entitled “Guide to AS9001:1994 for architectural engineering and design practices”. Whilst this standard is not directly quoted in mining legislation, one of the defences under Queensland Mining Law for example, is that due consideration has been given to relevant standards and guidelines. Clearly this is one such standard and the industry would possibly benefit by being aware of and adopting its general principles, which are not onerous and would almost certainly contribute to improved practice.

Basically the relevant legislation requires that credible geotechnical design is undertaken on a pro-active basis. Given this, the paper will now examine some of the other associated benefits to both operating mines and the industry in general.

**BENEFITS TO OPERATING MINES AND THE INDUSTRY IN GENERAL**

In addition to the benefit of compliance with the requirements of relevant mining law and the defence position that it provides, there are other tangible benefits of ensuring that credible engineering based design is undertaken for all geotechnical matters.

As an example, few of us would travel on commercial airliners if we suspected for one second that Boeing and Airbus Industries simply took their best guess, built planes that looked about right and then relied on a management plan should problems eventuate during operations. Whilst detailed and rigorous engineering design is no guarantee of success in any field of engineering, the likelihood of problems occurring must surely be significantly reduced in line with its use.
When the consequences of inadequate strata control are considered (safety, financial and reputation), the case for and resources required to undertake credible engineering design for strata control practices are easy to justify. It is simply not good business to take undue risks in the area of strata control and the quotation at the bottom of all of Dan Payne’s e-mails holds true:

“Nature cannot be tricked or cheated. She will give up to you the object of your struggles only after you have paid her price” (Napoleon Hill)

In this case, her “price” is the use of fit for purpose ground support/pillar dimensions etc. and the most reliable method of achieving this is surely through the use of credible and effective geotechnical design.

The other obvious benefit to operating mines of credible geotechnical design being undertaken by their own personnel are the professional developments that on-flow. There is nothing that focuses the mind of an engineer more than having to specify and fully document a geotechnical outcome in a transparent and auditable manner and then take responsibility for it by signing off the relevant documentation. This is the basis of real “engineering” as compared to the “geotechnician” type work undertaken in the on-going implementation of the strata management process (e.g. extensometry data collection and processing, borescoping, mapping etc.).

There is little doubt the coal industry will benefit hugely as more of its geotechnical personnel become proficient in the various areas of geotechnical design and apply them on an on-going basis at operating mine sites. It is also noted that this has the added advantage of having the designer fully involved with the construction process (listed earlier as a requirement of engineering design in general terms) which is rarely the case when the designer is a third party consultant.

WHY IS THE MINE SITE GEOTECHNICAL ENGINEER SO IMPORTANT TO THE FUTURE OF THE COAL INDUSTRY?

Some may argue and indeed have argued that there is no requirement for the mine site based geotechnical engineer to be able to undertake geotechnical design. His or her role is apparently to collect and collate the geotechnical data, run the strata management plan, manage secondary support contracts and simply draft support plans as they are required. It has even been suggested that geotechnical design is essentially beyond the mine site geotechnical engineer’s ability and that it should be solely the domain of those experienced persons who have come through a research and consulting background. The authors strongly disagree with this view and furthermore consider such a view would have long term detrimental effects.

If one examines the age and experience/background of senior geotechnical consultants working within the coal industry as well as the establishment of the various geotechnical consulting groups, some worrying trends emerge. Most senior geotechnical consultants are ex-ACIRL and it is now over 10 years since the geotechnical group within ACIRL essentially ceased operating. Most of the underground coal geotechnical consulting businesses were formed during the 1990’s. To the authors’ knowledge only one geotechnical business has been formed this current decade by any person(s) working in the Australian underground coal industry but not already actively operating as a geotechnical consultant.

However the most disturbing issue relates to an ageing group of senior geotechnical consultants/researchers within Australia; unfortunately most are on the wrong side of 50 and as best as the authors’ can ascertain very few (if any) less than 40. During a period of unprecedented growth in Australian underground coal mining that is predicted to continue for many years yet (albeit with cycles), the reality is that within the next 10 years many of the senior industry geotechnical personnel are likely to be retired or at the very least significantly reducing their geotechnical activities.

Clearly this is all indicative of an unsustainable situation and more to the point, an underground coal geotechnical knowledge “void” is potentially looming unless the industry recognises this and puts some succession planning in place.

The real problem is that the geotechnical training grounds are now gone or nowhere near as vibrant as they once were. The mining group at ACIRL is no longer in existence and large research establishments such as the Chamber of Mines in South Africa, the USBM (now NIOSH) and Bretby in the UK are either closed down or only a shadow of their former selves. Similarly the number of strata control Ph.D’s being awarded annually is at a very low level.

Whilst the authors are not suggesting that having a Ph.D is a requirement for a minesite strata control engineer, there is little doubt that one gains immeasurable benefit from spending three to six years studying an aspect of coal mine geotechnical engineering in the greatest detail and attempting to advance the associated level of knowledge and engineering. As well as then passing on that geotechnical knowledge and research “know-how” to others (i.e. mentoring).
With the traditional training grounds effectively gone and current strata control consulting companies being in huge demand and therefore doing much less geotechnical research than they used to, the question has to be asked as to what is the way forward?

**WHERE IS THE INDUSTRY UP TO AND SUGGESTIONS FOR THE FUTURE?**

Without doubt the industry has made significant advances in the area of professional development for strata control personnel, but clearly more needs to be done. Many of the mining companies have offered great support for the Strata Control Graduate Diploma offered by the UNSW and to-date more than 25 industry personnel have either completed the course or are in the process of completing it next year. This if nothing else is a clear indication that the role of the strata control engineer based at a mine site is becoming a specialist function (which it should be) requiring specific knowledge and training. The industry sees benefit in providing their personnel with appropriate qualifications and the individuals involved are prepared to opt for a career in this area.

However gaining a Graduate Diploma in Strata Control, a Masters in Rock Mechanics or even a Ph.D is not the end of the process. Competence in any discipline is a combination of skill and knowledge. A large parcel of knowledge can be gained in a classroom environment and tested under exam conditions. However as per the route to becoming a professional engineer or mine manager, a tertiary qualification is only a minimum requirement and the start of the process, not the end point. The competence requirement that is necessary to complement the knowledge component is that of skill, which is only borne of experience in the field. It is this aspect that now needs to be the focus of the industry so that mine site based strata control personnel become “engineers” in every sense of the word.

To achieve this, the following minimum requirements are suggested by the authors:

(i) Training and Support – mentoring/supervision in their role by suitably qualified and experienced personnel. Mentoring is a well recognised vital aspect of personal and professional development and needs to be intrinsic to our industry.

(ii) Design tools and Methods – in the same way that a ventilation officer would be unable to function effectively in their role without the availability of Ventsim for example, so a geotechnical engineer cannot function without fit for purpose design tools. A number are already available such as:

- UNSW pillar design method(s) – now available in Windows based software format as the “FOS Calculator” from Colwell Geotechnical Services
- ALTS 2006
- ADRS (Colwell 2004)
- NIOSH produced publications and assessment software
- ACARP project final reports that contain at least guidelines in a number of subject areas from research work (e.g. Frith and McKavanagh 2000, Hill 2006).

Clearly not every strata control design problem is currently covered by freely available design and assessment methods, but this situation is improving over time, in particular due to the efforts of the current Colwell Geotechnical Services industry project, whereby an analytical model for roadway roof stability is being combined with a large industry database of roadway roof stability experiences in an attempt to provide empirical/analytical design methodologies and software for mine roadways in longwall mining.

Even if a given mine site or mining company wishes to utilise third party providers for geotechnical design services, the need for mine site based geotechnical design capability still remains, as there is still the basis by which such consulting advice is either accepted or rejected by the mine site. It is well established that there is a fundamental need for decision-makers on engineering matters to evaluate third party consulting advice before implementation and make an informed decision as to whether to accept or reject it. Such a decision must be made on a credible basis, the undertaking of independent geotechnical design/evaluation by mine site personnel being an effective method of doing so.

The need for site based geotechnical assessment and design ability still remains and therefore, providing a full suite of design tools and associated professional development support should be one of, if not the top priority for the geotechnical fraternity within the Australian Coal Industry.

**SUMMARY**

The paper has attempted to provide a thought provoking discussion on the future importance of the mine site geotechnical engineer to the Australian underground coal industry and why it is vital that those persons have the skills and tools to be able to undertake credible geotechnical design as part of their job functions. That is not to say
that none do so at the current time, but the intensive use of third party geotechnical consultants indicates that their primary function is in the on-going implementation of the strata management plan.

The reasons as to why geotechnical design needs to be undertaken routinely at a mine site level are many and varied, but include:

- compliance with mining law
- prudent management of business risks
- the continuing professional development of mine site strata control personnel
- the “aging” geotechnical population within the industry and the non-availability of the traditional training grounds for such personnel
- the large amount of geotechnical design that needs to be undertaken often in a short timeframe, making it impractical for third party consultants to provide a comprehensive turn-key type design service.

The paper is not intended to be critical of the industry; in fact it fully recognises the significant advances that have been made in the past twenty years. Its main point is that the focus of strata control design practices within the industry must inevitably change from the current crop of consulting providers to mine site based engineers and that the industry will benefit significantly as a result. Whilst the industry has already made significant progress along that track, there is still much to do in order for the mine site based person to transit from the geotechnician to geotechnical engineering role, this being best achieved through the establishment of geotechnical design capability at the mine site level.

REFERENCES


POLYMER-BASED ALTERNATIVE TO STEEL MESH FOR COAL MINE STRATA REINFORCEMENT

C. Lukey¹, G. Spinks¹, E. Baafi¹, I. Porter¹ and J. Nemcik¹

ABSTRACT: The University of Wollongong in collaboration with the Australian coal mining industry has shown that a viable polymer-based alternative to steel mesh in underground roadway support applications can be developed to eliminate the use and handling of steel mesh. The feasibility of developing polymeric alternatives to steel mesh in underground roadway support applications has been established, the physical and material constraints to be endured by any new polymeric skin reinforcement system have been identified by measuring the mechanical properties of steel mesh, and materials that can be spray-applied have been identified. The study has also shown that polymer mechanical properties can be optimised to produce similar mechanical properties (modulus, yield stress, elongation-at-break etc) to steel mesh. The identified materials will allow the face support cycle to be fully automated, or at least remotely operated and installed, enabling the removal of personnel from the immediate face area, thus contributing to a projected substantial improvement in underground roadway development rates.

INTRODUCTION

Steel mesh has been used in underground coal mine roadways for some years. The main role of mesh is to provide passive confinement, especially in locations where poor ground conditions prevail, preventing fragments of rock and coal from falling from the roof and ribs in the spacing between reinforcing bolts.

Installation of mesh is a manual operation, and has been identified as a slow and inherently dangerous step in the roadway advancement process. Self-drilling bolting technology, with the potential for full automation, has been widely investigated over recent years; however the meshing process remains necessarily a manual operation.

A need for an alternative to steel mesh that can be installed automatically has been identified, which will allow the roadway support process to be fully automated, and thus take advantage of self-drilling bolt technology. The University of Wollongong in collaboration with the coal mining industry has been actively engaged in the search for a suitable alternative to mesh which has the following attributes:

- provides an effective skin confinement measure equivalent or superior to that of steel mesh;
- requires minimal human intervention in its installation;
- removes personnel from the immediate face area;
- enables higher underground roadway development rates to be achieved;
- is safe to use;
- is cost effective.

Thin Spray-on Liners

Thin spray-on liners (TSLs) are polymer-based materials that are used underground, and are mostly designed to provide secondary support in addition to steel mesh. Over 20 products are available in the market at the present time, and they fall generally into one of two material types: crosslinking polyurethane- or polyurea-based systems; and cement-reinforced water-dispersible systems based on ethylene-vinyl acetate copolymer (Espley-Boudreau 1999, Potvin et al 2004). The relatively narrow range of polymeric materials that form the basis of the majority of TSLs restricts the range of properties somewhat and hence the general applicability. Also, the use of cementitious additives in some TSLs improves the structural strength but unfortunately reduces the flexibility.

Prevailing Underground Conditions

The prevailing underground conditions are different in every mine. Roadway rib support practices can range from a single rib bolt per development metre and no mesh to three or more rib bolts and complete ceiling to floor meshing, depending on the structural soundness of the rib coal and the degree of ground movement experienced. Mesh usage in the roof, however, is commonly full width and continuous with a typical “square” bolting pattern. Roadway development practices range from “cut-and-flit” to bolting and meshing directly behind the continuous miner cutting head, again depending on the stability of the strata. In some mines gas drainage is an issue, whereas others have problems with mine water at low or high pH. Any new material will have to be able to be successfully applied, and provide the requisite level of long-term support, under these widely-varying conditions.

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This paper discusses some of the important issues associated with the replacement of steel mesh with a polymer-based alternative, and some of the strategies employed to deal with those issues.

**POLYMERIC SKIN CONFINEMENT PROPERTY REQUIREMENTS**

The desirable material properties of a polymeric skin confinement system include:

- able to be spray-applied without slumping;
- no toxic or irritant emissions during application, initial set or the development of full strength (curing);
- rapid initial set (seconds), and develops full strength over longer term (minutes to hours);
- good adhesion to coal, rock, roof and rib bolts prior to full cure;
- not sensitive to water, rock dust or coal dust;
- not pH sensitive;
- semi-permeable to water and gases;
- high strength, yet flexible (distorts within limits without rupturing);
- able to arrest or retard flaking and spalling of roof and ribs;
- strength enhanced by reinforcing fillers;
- light coloured;
- anti-static;
- fire retardant/intumescent.

A number of polymeric alternatives have been investigated that appear to have all of the chemical and physical property attributes required. The Flow Chart shown in Figure 1 summarises the material selection process. It can be seen that the selection process has essentially four stages, and progress is controlled by “yes-no” criteria:

- cure characteristics;
- flexural properties;
- viscosity and flow characteristics (rheology);
- environmental.

**Cure Characteristics**

The conceptual sequence of events when driving a roadway would be to cut the roadway using a continuous mining machine, install the confinement measure (whether steel mesh or some polymeric alternative), and then drill and install the bolts. In order to achieve this in minimum time, a polymeric confinement material would need to progress from liquid to solid in a matter of a few seconds (referred to as “cure”) after spray application.

Cure chemistry to a large extent governs the speed of conversion from the sprayed liquid to a solid polymer, and the type of emissions (if any). There is a range of cure chemistries that are rapid-cure (several seconds) with no small molecule emissions, or that emit only water.

Polymer crosslinking (cure) is commonly a two stage process involving gelation followed by vitrification (Chang & Chen 1987, Martin et al 2000, Cook et al 2001). Monitoring of cure can be achieved using a differential scanning calorimeter (DSC), which measures heat flow as a function of time and/or temperature. A typical DSC measurement of cure is shown in Figure 2. After a period of no heat flow (induction period) which can last from seconds to hours depending upon the promoters and accelerators used, the material forms a gel (first stage cure) and a sudden exotherm occurs. At this point the material is dimensionally stable but not structurally sound or strong, and the material could then be drilled and bolted. Over the next period of time, which could also range from seconds to hours, vitrification occurs (second stage cure - the material becomes a glass) and the material attains structural strength.
Figure 1 - Flowchart for selection of candidate materials
Flexural Properties

As movement occurs in underground strata, flexural loads will develop in any containment measure, so it is important to understand the flexural strength of both mesh and the polymeric replacements for mesh. Flexural strength is best measured in the laboratory by subjecting test specimens to a 3-point bend test. In this test, a rectangular beam of the test material is subjected to a bending load, and the flexural strength is calculated from the maximum load reached according to the following equation:

$$\sigma_{FS} = \frac{3FL}{2bh^2}$$

where

- $\sigma_{FS}$ = flexural strength (MPa)
- $F$ = maximum load reached (N)
- $L$ = distance between supports (mm)
- $b$ = sample width (mm)
- $h$ = sample thickness (mm)

If the tensile modulus ($E$) of the material is known, it is also possible to calculate the maximum deflection ($\delta$) that would occur before failure:

$$\delta = \frac{FL^3}{4Ebh^3}$$

The flexure behaviour of a number of reinforced polymers was measured at a constant deformation rate of 2mm/min, and the results are shown in Figure 3. Note that none of the materials exhibited catastrophic brittle failure. Instead, a gradual loss of strength was observed, due to the presence of the reinforcing filler. As shown in the Figure, some formulations were strong but too brittle, whereas others were less strong but more flexible. Hence a number of polymers have been prepared and tested that display a wide range of flexural properties. Detailed modelling of steel mesh properties will help to identify the polymer that exhibits the best properties for the application.

Rheology

It is most likely the new material will be spray applied, so an acceptable viscosity range will need to be defined and measured. In addition, the material will need to be applied without slumping, so the rheology is important. A thixotropic agent may be included in the formulation to prevent slumping, and the amount will need to be determined experimentally. Both of these can be addressed by the use of viscometry measurements.

Environmental Characteristics

Environmental issues are such things as temperature stability, pH tolerance, fire retardancy, toxicity and anti-static properties. For a crosslinked polymer, the glass transition temperature is the temperature at which the material converts from a high modulus glassy solid to a low modulus rubbery solid. In order to have stability and strength under prevailing underground conditions, the glass transition temperature of the polymer would need to be in excess of about 50°C.
Figure 3 - Flexure Tests on Candidate Reinforced Polymers

As noted above, in some mines water is a problem. The pH of mine water can vary from 2 to 10, so the new material must be relatively unaffected by the pH of the mine water, in terms of physical properties and adhesion to wet coal.

Fire retardancy, anti-static properties and toxicity are all the subject of ASTM and Mine Safety methods.

CRITICAL ISSUES GOVERNING MATERIAL SELECTION

Results to date have shown that some reinforced crosslinking polymers can have appropriate mechanical properties as steel mesh replacements. A number of issues, however, need to be addressed before any potential material could be considered suitable for underground use. In order to identify these issues, a comprehensive product and process risk assessment has been carried out, which has identified the following critical issues:

- application quality control;
- health issues such as toxicity and irritancy during application and in the finished material;
- appropriate product mechanical properties;
- longevity;
- material must be anti-static and not propagate fire;
- no adverse effect on coal preparation plant or coal clearance systems.

Application Quality Control

As noted above, ground conditions are different in every mine. Some mines may require a continuous skin confinement measure of uniform thickness, whereas others may require a thick band in the vicinity of the bolts and thinner sections in between the bolts. There may be yet other requirements at other sites. Also, freshly-cut coal does not present a smooth surface. For these and other reasons, application quality control is seen as an important issue. Application of the polymeric mesh replacement will be automated, so the application technology will need to be able to apply the material in a manner consistent with the prevailing ground conditions.

Health Issues

The major health issue in relation to a polymeric material being used underground in large quantities is the possibility of toxic or irritant emissions during installation and/or in service. This is particularly relevant where a chemical reaction occurs to effect cure of the polymer (as in the present case). In our selection of polymer and cure chemistries, we have investigated only those systems that have no condensation product during cure, or emit only water, however the issue remains of possible volatile chemicals in confined environments prior to cure. Our investigations have identified raw materials that are very low volatility, thus would not present a toxic or irritant hazard.
Mechanical Properties

A goal of this work is to develop a polymer-based material that has equivalent or superior mechanical properties to steel mesh. In order to determine the appropriate mechanical properties of a steel mesh replacement, it is first necessary to determine the properties of steel mesh itself. This can be achieved by a number of means. Steel mesh is constructed using drawn low carbon steel wire welded in a square mesh pattern. Mesh is typically 4% steel by volume, thus a very rough estimate of the tensile properties of mesh in the direction of the wire can be made based upon measured properties of the wire. Typical tensile properties of steel wire and mesh, and a reinforced polymer of the types described above, are shown in Table 1. It can be seen that, with the exception of failure strain, the reinforced polymers possess tensile properties similar or superior to steel mesh. Geotechnical modelling of the role of steel mesh underground will lead to a more robust and realistic system for the determination of the mechanical properties to be endured by any polymeric replacement for mesh.

Table 1 - Typical Tensile Properties of Steel Wire, Mesh and Reinforced Crosslinking Polymer

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>Low Carbon Steel Wire</th>
<th>Low Carbon Steel Mesh</th>
<th>Crosslinking Polymer 30-50% Fibre</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (GPa)</td>
<td>205-215</td>
<td>8</td>
<td>10-17</td>
</tr>
<tr>
<td>Yield Strength (MPa)</td>
<td>500-600</td>
<td>20-24</td>
<td>25-55</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>500-600</td>
<td>20-24</td>
<td>30-70</td>
</tr>
<tr>
<td>Failure Strain (%)</td>
<td>4-6</td>
<td>4-6</td>
<td>1-2</td>
</tr>
</tbody>
</table>

A second issue in relation to mechanical properties of the mesh replacement is adhesion of the cured material to strata. The current plan is to use a material that will achieve initial cure in a few seconds (dimensional stability, but perhaps not full strength) and will then be bolted through. Some adhesion of the uncured (wet) material to the rock or coal strata would be advantageous, and adhesion of the cured material is seen as a potential additional skin reinforcement mechanism. A quantitative adhesion test has been developed which allows the measurement of adhesion strength of an applied reinforced polymer to a number of different rock types and coal under a variety of conditions (wet, dry, dusty, low pH etc).

Longevity

In longwall mining operations, gate roads are in use during the extraction of two longwalls, first as a main gate and then as a tail gate. A skin confinement measure would thus need to remain in good condition for this entire duration, which may be 2 to 3 years.

The major mechanisms by which deterioration of polymer properties can occur are degradation during processing, thermal degradation, weathering, and environmental stress cracking. Processing and thermal degradation generally occur as a result of exposure of the polymer to elevated temperatures either during synthesis, manufacture of an article from the finished polymer, or in service. In the mine environment temperatures tend to remain very stable, so thermal degradation is unlikely.

Weathering, as the name implies, is degradation as a result of exposure to weather. The most important cause of weathering is exposure of the polymer to solar ultraviolet light in the presence of atmospheric oxygen, a process known as “photo-oxidation” (Rabek 1995). Oxygen itself can have no effect on a polymer in the absence of UV light or excessive heat, hence polymer weathering is very unlikely underground.

Environmental stress cracking involves crack initiation, growth and ultimate failure by the combined action of a tensile stress and an environmental liquid or gas. As ground movement occurs constantly in mine roadways, the stress experienced by a skin confinement measure would also be constantly changing. Some polymers are very susceptible to environmental stress cracking, and the environmental liquid causing the degradation could be as otherwise-innocuous as water. Other polymers are much less susceptible to the phenomenon. The types of polymers being investigated for skin confinement are not known to be susceptible to environmental stress cracking. Accelerated testing of any potential candidate material will have to be carried out in order to ascertain that the polymer will have the required longevity in service.

Anti-static and Fire Retardancy

Polymers are typically electrical insulators, and as such can accumulate static electricity generated by friction as a result of air flow across surfaces. A build-up of static electricity underground can lead to a spark discharge, which can in turn lead to fire. Steel mesh is able to conduct static electricity safely away by way of the earthing effect of the bolts, however polymers intrinsically are not able to do this. The addition of anti-static additives may alleviate this problem. Several anti-static additives for polymer systems are available (Grob & Minder 1999), however the solution may be simply the addition of coal dust. A significant proportion of coal is graphitic carbon, which is electrically conductive. The amount of coal dust or other additive to give the conductivity required by legislation will have to be experimentally determined.

One major advantage of steel mesh over any polymeric alternative is that it is non-flammable, whereas polymers tend to be at least combustible if not spontaneously flammable. Fire retardancy can be incorporated into a polymeric formulation either by using fire-retardant monomers, which tend to be brominated materials, or by use of
a fire-retardant additive. The use of brominated raw materials would increase the cost enormously, so fire-retardant additives are a better option. Such additives are of two types: fire-suppressant; and intumescent.

Fire suppressant additives typically decompose when heated to produce carbon dioxide or other non-combustible gas, which then suppresses the flame (Biswas et al. 2007 & references therein). Intumescent materials carbonise and swell when burnt, removing the seat of the fire from the surface (Ma et al. 2007 & references therein). Either type of additive would be suitable for the present application.

**Impact on Downstream Processing**

The final issue relates to the effect of the new product on coal preparation plant and coal clearance systems. The major consideration is the possibility that the polymer may become tangled in the shearer head, however the stiffness of the reinforced polymer makes this very unlikely.

**CONCLUSIONS**

A polymeric alternative to steel mesh for underground coal mine roadways offers numerous advantages over mesh. There are, however, also a number of issues which will need to be addressed before any polymeric material could be used in this capacity. Recent research has shown that a viable polymer-based alternative to mesh can be developed which overcomes all or most of the critical issues described.

Future work will be focussed on the further development of suitable materials, especially in relation to environmental issues such as pH sensitivity, and the control or prevention of toxic or irritant emissions during application and cure. In addition, a comprehensive geotechnical study will be undertaken into the role of steel mesh in roadway support, and this will further guide the development of a polymeric alternative.

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CORROSION PROTECTION OF ROCK BOLTS BY EPOXY COATING AND ITS EFFECT ON REDUCING BOND CAPACITY

Mahdi Moosav\(^1\) and Sepideh Karimi\(^2\)

ABSTRACT: Corrosion protection of fully grouted rock bolts has been the subject of considerable research in recent years. Corrosion protection is studied focusing on quantitatively determining how much encapsulation coating affects the bolt/resin bond capacity. Resin coating results in reduction of rib height and in turn causes a decrease in interlocking effect with the grout annulus. The laboratory tests performed have shown that there was a wide range of reduction in bonding strength (from 5 to 40%), depending on the type of the bolt and media in which the bolt had been installed. The reduction of rib height was also responsible for lower lateral dilation during bolt pullout tests. This effect will make the confining medium become an important parameter, since higher confining medium results in higher confining pressure on the bolt surface which in turn, controls the bond capacity.

INTRODUCTION

For years, rock bolts have been a common method for ground reinforcement both at underground as well as surface rock structures. Effectiveness and ease of installation has been the main two advantages of this active support method as opposed to the usual passive ways of supporting broken rock. Most of the bolts are made out of steel which makes them good candidates for corrosion. Although fibreglass bolts have recently emerged into the market for special applications, nevertheless steel bolts have still remained the dominant type of rock bolt in daily practice.

One of the main problems about rock bolts, especially underground, is corrosion. The main causes of this problem are underground water, humidity, stray currents and chemical interaction between the surrounding media and the steel. Grounds with sulphur content, when interacts with water, can produce strong acids which quickly reduce the effective diameter of the bolt. This problem in certain circumstances becomes so severe that can cause failure of the reinforcement.

One of the temporary methods to overcome this problem is to apply a corrosion resistant coating on the surface of the bolt. Epoxy resin is one of these materials which have been widely used due to its relatively low price. Although this can be assumed only a temporary solution, but in many occasions, the lifetime of the tunnel which these reinforcements are to be used in is also short therefore their application is justifiable. For higher required life times, stronger protections are required. Double Corrosion Protection systems (DCP) utilizes a high density polyethylene tube as well as a layer of cement around the bolt as two corrosion protection layers, to ensure higher corrosion securities.

One of the causes of epoxy coating is reduction of the effective rib height in the bolts which in turn, can reduce the bond capacity due to reduced interlocking effect with the grout annulus surrounding the bolt. The present paper tries to find a quantitative answer to this general feeling about reduced bond capacity.

ROCK BOLTING IN MINES

Rock masses contain natural discontinuities which may cause stability problems, therefore most underground openings need to be stabilized to maintain their integrity during their service life. As stated by Hoek and Brown (1980), “The principal objective in the design of underground excavation support is to help the rock mass to support itself”. The best way to achieve this is through the use of reinforcement (i.e. rock bolt) to help maintain the load-carrying capability of rock masses near excavation boundaries.

Beyl (1945) reported an early use of bolts in a longwall mine in 1912. The bolt was made out of wood and was used to prevent small pieces of rock from falling between the face and the main support system. Littlejohn and Bruce (1977) reported that the first use of rock anchors was in Cheurfas Dam, Algeria in 1934. Due to the success of bolting, fundamental studies on the bolting action were started by Rabcewicz (1955) and was continued by Panek (1956a, b,c,1962a,b) supported by U.S. Bureau of Mines. This research led to the concepts of suspension and beam building effects for bolts in bedded mine roofs. The arching effect of bolts was pointed out by Evans (1960). In jointed rocks, the importance of limiting displacement as the key parameter of the bolting action was explained by Palmer et al. (1976). This is because the opening of joints during excavation decreases the strength of rock due to the associated softening effect. This concept forms the basis of present pre-reinforcement concepts.

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It was noted that pre-placement of bolts can decrease the deterioration of the internal rock mass strength resulting from joint dilation.

Rock bolting is currently a usual practice in most of the coal mines due to the limited required length for the reinforcing rock layers and the request for high installation speed as a prerequisite for increasing production time. Most of the rock bolts use resin capsules as a bonding agent to the rock which facilitates faster process and reduces time for the whole supporting cycle.

CORROSION MECHANISM

Corrosion is defined as defect on material (usually metals) properties due to their interaction with the surrounding media. By this definition, wear, abrasion, scratch and fatigue which have mechanical cause are excluded. It is worth noting that the word “rust” is used only for Iron which is an interaction with water and Oxygen. In another words, other metals will corrode but do not rust.

The main chemical mechanism in steel corrosion is as follows:

$$Fe + H_2O + \frac{1}{2}O_2 \rightarrow Fe(OH)_2$$

So if anyone of the main three components (steel, water and oxygen) does not exist, the corrosion would not happen.

In this mechanism, some parameters can have accelerating effects which the most important ones are as follows.

- Temperature: Usually the higher the temperature, the faster the corrosion would be. The hotter points in a material are usually more anodic than the other points so cause accelerated corrosion locally.
- Difference in galvanic potential: When two metals with different galvanic potentials are close to each other, the metal with higher galvanic number acts as anode and corrodes faster so protects the other metal from corrosion.
- Surface smoothness: Metals with rough surfaces usually corrode faster than shiny surfaces.
- Stress: When a material is under tensile stress, it corrodes faster which is believed to be due to the micro cracks generation in the metal. Corrosion will accelerate if the stress level is higher, especially if it is close to the material’s elastic limit.

TEST SETUP AND SAMPLE PREPARATIONS

The bolts used for tests consisted of two types, i.e. 28mm rebar and 28mm continuous thread bar from Dywidag company. The bond length in each sample was 15 cm and the water:cement ratio used for the grout annulus was 0.4. Some of the bolts were covered by epoxy resin while some others were left uncoated to enable comparison of bond reduction. The number of the bolts used in each test class is shown in Table 1.

<table>
<thead>
<tr>
<th>Type of confining pipe</th>
<th>Type of bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC</td>
<td>CT bar Φ28 with Epoxy</td>
</tr>
<tr>
<td></td>
<td>CT bar Φ28 without Epoxy</td>
</tr>
<tr>
<td></td>
<td>Rebar Φ28 with Epoxy</td>
</tr>
<tr>
<td></td>
<td>Rebar Φ28 without Epoxy</td>
</tr>
</tbody>
</table>

Since the bolts are usually installed in 63.5 mm (2.5 inch) diameter borehole, the laboratory test was designed so that the bolts become surrounded by the same size mould. To confine the bolts, they were put in pipes with internal diameter of 63.5 mm (2.5 inch) and the space between the bolt and the pipe was filled with Portland Cement grout. This test was carried out “constant confining stiffness” condition, meaning that during pull test, the generated pressure at the outer surface of the grout will vary as a function of generated dilation due to ribs. These pipes were made of Steel, Aluminium and PVC, to simulate different rock mass qualities in the laboratory. To associate each pipe to a rock mass with known quality (i.e. $E_m$) equation (1) can be used.

$$k_r = \frac{2E}{(1+\nu)} \left[ \frac{d_o^2 - d_i^2}{d_i(1-2\nu)d_i^2 + d_o^2} \right]$$ (1)
In this equation \( k_r \) is the radial stiffness of a pipe with \( d_i \) and \( d_o \) as its inner and outer diameters and \( E \) and \( v \) as the elastic properties of the pipe material. Table 2 shows the radial stiffness of the various pipes used as moulds.

![Figure 1 - Sample preparation.](image)

Table 2 - Radial stiffness of the pipes used as mould.

<table>
<thead>
<tr>
<th></th>
<th>( E ) (GPa)</th>
<th>( v )</th>
<th>( d_i ) (mm)</th>
<th>( d_o ) (mm)</th>
<th>( k_r ) (MPa/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>200</td>
<td>0.25</td>
<td>58.4</td>
<td>45.5</td>
<td>2110.33</td>
</tr>
<tr>
<td>Aluminium</td>
<td>72</td>
<td>0.25</td>
<td>59.6</td>
<td>50.0</td>
<td>504.81</td>
</tr>
<tr>
<td>PVC</td>
<td>3</td>
<td>0.32</td>
<td>62.44</td>
<td>53.3</td>
<td>19.17</td>
</tr>
</tbody>
</table>

For a borehole drilled with radius \( r \) in a rock mass having deformation Modulus and poison's ratio equal to \( E_r \) and \( v \) respectively, the radial stiffness is:

\[
k_r = \frac{E_r}{(1 + v)r}
\]

therefore the steel pipe, for example, used in the tests is equivalent to a rock mass having deformation modulus equal to 83 GPa since;

\[
2110.33 = \frac{E_r}{(1 + 0.25)31.5} \quad \text{or} \quad E_r = 83000 \text{ MPa}.
\]

At the time of pouring cement annulus around the bolts, cylindrical samples were taken from the grout and their mechanical properties were determined after 28 days.

After the grouted bolts were left to cure for 28 days, they were put in the testing setup and were pulled for determination of bond capacity. This consisted of a 600 kN hollow ram jack activated via a hydraulic pump. The loading force is determined using an electrical load cell and the bolt displacement during pull was measured using an electrical displacement sensor (LVDT) with 0.01 mm accuracy at the exit point of the bolt. The whole system was connected to a data acquisition system (DAS) for automatic data collection and storing.
Figure 2 - Grout samples poured for mechanical properties tests.

Figure 3 - Test setup with the hollow ram jack and DAS system for data collection.

TEST RESULTS

Figures 4 through 9 show the comparative results for pullout force for each type of bolt with and without epoxy coating. Note that each graph is the average of the number of tests in that category that was mentioned earlier in Table 1.

Table 3 has summarizes the pull force results for the above tests.
Table 3 - Comparison between the maximum pullout forces for the tested samples

<table>
<thead>
<tr>
<th>Type of bolt</th>
<th>Type of confining pipe, peak load and pull force reduction due to epoxy coating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td></td>
<td>Peak Load (kN)</td>
</tr>
<tr>
<td>CT bar Φ28 with Epoxy</td>
<td>110</td>
</tr>
<tr>
<td>CT bar Φ 28 without Epoxy</td>
<td>181</td>
</tr>
<tr>
<td>Rebar Φ28 with Epoxy</td>
<td>152</td>
</tr>
<tr>
<td>Rebar Φ28 without Epoxy</td>
<td>161</td>
</tr>
</tbody>
</table>

Figure 4 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in a steel pipe.

Figure 5 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in an Aluminium pipe.
Figure 6 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in a PVC pipe

Figure 7 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in a steel pipe
Figure 8 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in aluminium pipe.

Figure 9 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in a PVC pipe.
DISCUSSIONS AND CONCLUSIONS

According to the obtained results, the following conclusions can be made:

1. In all results, higher bond capacities are obtained from bolts without epoxy coatings regardless of the bolt type. This reduction is ranging from 5 to almost 40 percent in different bolts.
2. Reduction of bond capacity due to epoxy coating is believed to be due to reduction in effective rib height and creation of a more smooth bolt so the frictional properties of the bolt-cement interface is reduced.
3. Higher bond capacities are obtained from pipes with higher radial stiffness i.e. Steel, Aluminium and PVC respectively.
4. The wavy form of pull curves is notable in these tests and is believed to be from the passage of ribs through the sound and crushed cement regions. Correlation of wave lengths and rib spacing confirms this finding.
5. After increasing shear displacement, the shear face becomes more and more smooth which explains the decaying trend of each load peak.
6. Lower bond results are obtained from CT bars compared to rebar which are explainable by existence of two smooth sides on the CT bolts. These area has no ribs hence reduces the frictional properties of the bolt surface during pullout.

ACKNOWLEDGEMENT

This research was funded by the Research deputy of The University of Tehran and tests were performed at Rock Mechanics laboratory which both supports are highly appreciated and acknowledged.

REFERENCES

OPTIMISATION OF THE BOLT PROFILE CONFIGURATION OF LOAD TRANSFER ENHANCEMENT

Naj Aziz¹, Hossien Jalalifar¹, Alex Remennikov¹, Shane Sinclair¹, and Andrew Green¹

ABSTRACT: Both bolt profile shape and profile spacing (rib spacing) have been found to influence the bonding capacity of the grouted rock bolt. The bolt surface profile configuration has greater importance to rock bolt than the steel rebar used in civil engineering construction, because the rock bolt is subjected to greater dynamic loading than the steel rebar. The increased bonding capacity of bolts is important when supported ground is either heavily fractured, faulted or the supported ground is of soft formation, typically that of coal measure rocks. Past laboratory studies have identified the bolt profile spacing as of significant relevance to bolt resin rock bonding increase, however, no attempt has been made to determine the optimum spacing between the bolt profiles spacing. Accordingly, a series of laboratory tests were carried out on 22 core diameter bolts installed in cylindrical steel sleeve. The study was carried out by both push and pull testing. The push testing was carried out in 150 mm long sleeves while the pull testing was made in 115 mm long sleeves. Profile spacing tested include, 12.5, 25.0 mm, 37.5 mm and 50 mm lengths. The profile spacing of 37.5 mm wide was found to provide the optimum bearing

INTRODUCTION

Rock bolts used for rock formation reinforcement differ in function from the steel ribbed rebar used in concrete reinforcement in building construction. The reinforcing effect of a grouted bolt is by the longitudinal and shear displacement in the rock mass. Thus the load transfer capacity of the bolt is governed by the shear strengths developed between the rock/grout and the grout/bolt. The bonding capacity of the bolt is in turn is influence by the bolt profile configurations. The profile configuration is defined by the rib profile shape, and height, angle of wrap and spacing or distance between the ribs.

Blumel (1996) was the first to report on the influence of profile spacing on load transfer capacity of the bolt. Figure 1 shows the results of a test of a particular rock bolt type with different distance or spacing between the ribs. The tests were undertaken in a specially constructed laboratory apparatus consisting of a 500 mm long steel pipe filled with concrete. The concrete had a central hole of diameter twice the bolt diameter. The bolt was anchored in the concrete cylinder using cementitious grout and the bolt pull-out tests were carried out with different displacement rates, applied to the bolt right from the installation. Blumel reported pull tests on different profile spacing, of 13.7 mm, 27.4 mm and 54.8 mm, and with pull-out tests values increasing with increased widening of the spacing respectively as shown in Figure 1. The tests were carried out with respect to time of loading up to 32 hours, with the pullout displacement rate of 0.72 mm/hr. The study clearly demonstrated that the pull-out force of the bolt differed greatly by varying the rib distance. No effort was made by the researchers to investigate the optimum spacing of the profiles for optimum bolt transfer capacity. Blumel, Schweiger and Golser (1997) reported on the final element modelling of the bolts with different profile spacings. Their study supported the experimental laboratory findings, which, as shown in Figure 2, clearly demonstrated that higher stresses with more significant peaks being developed in the case of the bolt with wider spaced ribs as compared to the small rib distance.

Aziz, and Day (2002) studied bolt profile spacing and load transfer conditions under constant normal stiffness (CNS) conditions under different confining pressures. The study confirmed the existence of changes in the load - displacement profiles with respect to bolt surface profile configurations. Moosavi, et al, (2005) also studied the profile configurations in cementitious grout, leading to similar conclusions. Aziz and Webb (2003) extended the study on profile configurations to include push testing of bolts installed in cylindrical steel tubes, 75 mm long and 17 mm in internal diameter. The tests were made using chemical resin instead of cement. Aziz and Jalalifar (2005 and 2006) extended this study to include both push and pull tests. Longer steel sleeve lengths greater than 75 mm were also used. 75 mm long steel sleeves were found to be of insufficient length to provide adequate number of profiles encapsulated in it to allow credible and meaningful test results. Aziz and Webb (2003) work concurred with the findings of the Blumel study on the effect of profile spacing on load transfer capacity of the loaded bolt.

There has been no reported attempts made to optimise the true bolt profile configurations for optimum load transfer capacity determination, and accordingly this paper represents the continuation of the work undertaken by the mining group at the University of Wollongong (UoW),and describes the laboratory testing of bolts in long steel sleeves which is aimed to address the profile spacing optimisation.

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Figure 1 - The load / displacement profiles of different profile spacing bolts. Bolts were installed in a cementacious grout. The rate of loading being at 0.72 mm/hr

Figure 2 - Axial stress developed on bolts of two different spaced profiles

EXPERIMENTS

In order to obtain better understanding of the influence of increased profile spacing and bolt load capacity, two series of tests were carried out on bolts in cylindrical steel sleeves. In the first series of tests, bolts with different profile spacing were push tested in 150 mm steel sleeves, while the second set of tests were made under pull conditions using 115 mm steel sleeves.

Table 1 shows a summary of the profile dimensions for all the bolt types that were tested. Wider profile spacings were achieved by grinding various profiles. Bolts with widened spacings were labelled G1, G2 and G3 with one, two and three profiles removed respectively. The respective spacings were 25 mm, 37.5 mm and 50 mm. No tests are reported for Bolts T1 and T3 as the comparative tests were reported previously by Aziz, Jalaifar, and Conclaves (2006).
Table 1 - Profile configurations of various bolts

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>T1</th>
<th>T2</th>
<th>T3</th>
<th>T2 Bolt Modified</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>G1</td>
</tr>
<tr>
<td>Profile Spacing (mm)</td>
<td>12.50</td>
<td>12.50</td>
<td>25.00</td>
<td>25.00</td>
</tr>
<tr>
<td>Profile Height (mm)</td>
<td>1.00</td>
<td>1.35</td>
<td>1.20</td>
<td>1.35</td>
</tr>
<tr>
<td>Average Profile Width (mm)</td>
<td>2.25</td>
<td>2.75</td>
<td>3.75</td>
<td>2.75</td>
</tr>
<tr>
<td>Profile Angle</td>
<td>22.5°</td>
<td>22.5°</td>
<td>22.5°</td>
<td>22.5°</td>
</tr>
</tbody>
</table>

Push test

Figure 3 shows a general view of push testing of bolts of different profiles in 150 mm steel sleeves. The procedure for testing is described elsewhere (Aziz, Jalalifar, and Concalves (2006). The tests were made in a 50 tonne capacity servo-controlled Instron Testing Machine. The encapsulation medium was a reinforced polyester resin grout BPI Mix and Pour resin. The resin had curing time of 60 minutes. The UCS strength of the resin was in the order of 70 MPa after seven days, the shear strength was 16 MPa, modulus of elasticity of 12 GPa, and stiffness value after 14 days was around 75 kN/mm.

As can be seen from the test result in Figure 3, the loading capacity of the bolt increased with increased profile spacing. However, the highest loading capacity was achievable with profile spacing of 37.5 mm rather than 50 mm rib profile spacing. The loading of 37.5 mm spaced bolt was halted as the unencapsulated bolt section began to bend. For the indicated final level push load of 425.8 kN shown for 37.5 mm spaced profiled bolt (Bolt Type T2 G2) in Figure 3, this is 7% greater than the maximum load achievable of Bolt Type T2 G3 of 50 mm profile spacing, and is 16% greater than of Bolt T2 G1 of 25 mm profile spacing, as shown in Table 2. The loading capacity of T2 G2 bolt is 97.5% greater than the original Bolt Type T2, with 12.5 mm profile spacing. It should be noted that the differences between the load bearing capacity between the 25 mm profile spaced Bolt Types T2 G1 and T3 is attributed to the surface roughness of the Bolt Type T2G1, which was resulted from the removal of the profile from Bolt Type T2. The effect of bolt surface roughness on the load bearing capacity of a bolt was previously reported by Aziz and Webb (2003). It is also equally true that the variations between the load bearing capacity between Bolt Types T2G2 and T2G3 could have been influenced by the increased surface roughness of Bolt Type T2G3, nevertheless, the bearing capacity of Bolt Type T2G3 is significantly higher than the T2G3.

Table 2 - Changes in the load capacity of different profile spaced bolts with respect to Bolt Type T2 in push testing (encapsulation length 150 mm)

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Profile spacing (mm)</th>
<th>Average. applied load (kN)</th>
<th>Increase in load with respect to Bolt Type T2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt Type T2</td>
<td>12.5</td>
<td>215.6</td>
<td>-</td>
</tr>
<tr>
<td>Bolt Type T2 G1</td>
<td>25</td>
<td>365.9</td>
<td>69.7</td>
</tr>
<tr>
<td>Bolt Type T2-G2</td>
<td>37.5</td>
<td>425.8</td>
<td>97.5</td>
</tr>
<tr>
<td>Bolt Type T2-G3</td>
<td>50.0</td>
<td>398.2</td>
<td>84.9</td>
</tr>
</tbody>
</table>
A number of preliminary tests were made to study the bonding capacity in 150 mm sleeve encapsulation under pull-out conditions, and this was discontinued as the pull-out load exceeded the elastic limit of the steel rebar bolt. This was particularly true when testing bolts greater than 25 mm profile spacing. Noting that both Bolt Type T2-G1 and T3, with rib spacing of 25 mm, had the yield load of 250 kN and ultimate tensile strength of more 330kN.

Accordingly the next series of tests were carried out under pull testing conditions with the encapsulation length of the steel sleeve reduced to 115mm as shown in Figure 4. Figure 5 shows the load displacement profiles for four profile spacing of 12.5 mm, 37.5 mm and 50 mm respectively. Also included in Figure 5 are the load displacement graphs of 50 mm profile spacing prepared from Bolt Type T3. The difference between the profiles configurations of various bolts are as per described in Table 1.
As can be seen from Table 3, the bonding capacity or the peak load of the bolt with profile spacing 37.5 mm is, once again, greater than the 50 mm profile spacing. In this batch of tests the maximum pull out force was within the steel rebar yield load, thus there were no significant changes in bolt diameter, as would have happened in push testing.

When compared to the standard Bolt Type T2 (profile spacing 12.5 mm), all other bolts experienced an increase in the average maximum peak load capacity. The Bolt Type T3 with the modified profile spacing of 50 mm experienced an average increase of 41% in pull load of 215 kN against Bolt Type T2 load of 152.23 kN. Of more significance was the increase in loading capacity of both Bolt Types T2G2 and T2G3 respectively. The average peak load of the T2-G2 bolts with profile spacing of 37.5 mm was 69% greater than that of the standard Bolt Type T2. Similarly for the Bolt Type T2G3, with 50.0 mm profile spacing, there was an increase of 61% with respect to Bolt Type T2.

![Load displacement results of different configuration bolts in pull testing](image)

**Figure 5 - Load displacement results of different configuration bolts in pull testing**

**Table 3 - Changes in the load capacity of different profile spaced bolts with respect to Bolt Type T2 in pull testing (encapsulation length 115 mm)**

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Profile Spacing (mm)</th>
<th>Average Pull load (kN)</th>
<th>Change (increase) in load with respect to Bolt Type T2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt Type T2</td>
<td>12.5</td>
<td>152.23</td>
<td>-</td>
</tr>
<tr>
<td>Bolt Type T3 G1</td>
<td>25</td>
<td>215.23</td>
<td>41</td>
</tr>
<tr>
<td>Bolt Type T2-G2</td>
<td>37.5</td>
<td>256.55</td>
<td>69</td>
</tr>
<tr>
<td>Bolt Type T2-G3</td>
<td>50.0</td>
<td>244.72</td>
<td>61</td>
</tr>
</tbody>
</table>
FUTURE WORK

Additional tests must be undertaken by pull testing in the laboratory concrete block, the field test, double shearing test and dynamic drop test.

Preliminary double shearing tests carried out by the authors have lead to inconclusive results. These tests were made in the same share box as that reported by Aziz, Pratt and Williams (2003). Suffice to say that the shearing characteristic of the wider profile bolts with spacing greater 25 mm and greater, were of similar characteristics as that reported by Aziz, Pratt and Williams. Future tests will be carried out in a much larger shear box, as shown in Figure 6.

The load drop test (Figure 7) is aimed to subject the bolt to impulsive dynamic loading. The objective is to examine the performance of different bolts under different dynamic loading conditions. The dynamic shearing characteristics will be examined under a range of impulse loading conditions by varying the drop height of a 600 kg anvil onto a test sample in a double shear box, thus enabling variable amounts of impact energy to be imparted to the test specimens.
CONCLUSIONS

It is abundantly clear from this study and from overseas that, the bonding capacity of the bolt increases with increased profile spacing. The profile spacing 37.5 mm was found to be the optimum spacing width with the particular type of bolt (with given profile orientation and shape).

For the wider spaced bolts to be assured of its performance in reality, tests must be extended to pull testing in the field as well as carrying out double shearing tests to examine the effect of latter forces in shear.

ACKNOWLEDGEMENT

The authors wish to accord their appreciation to Minova Australia, DSI-Dywidag Mining Systems, Australia, and Jennmar Australia for providing material to conduct the tests reported in the is paper.

REFERENCES

AN EMPIRICAL APPROACH IN PREDICTION OF THE ROOF ROCK STRENGTH IN UNDERGROUND COAL MINES

S.R. Torabi¹, F. Sereshki², M. Zare³, M. Javanshir³

ABSTRACT: In study of the behaviour of roof strata in underground coal mines the strength of the roof rock, particularly, the unconfined compressive strength (UCS) plays a significant role. Application of simple tools in assessment of the rock strength has been practiced by many researchers one of which being Schmidt hammer. Due to its portability, easiness in use, rapidity, low cost and its non-destructive procedure of application, it is among the most popular tools in this respect. Application of this tool in prediction of the roof rock strength, in a new context, is the aim of this research work.

A comprehensive review of the literature revealed that most of the empirical equations introduced for determination of the unconfined compressive strength of rocks based on the Schmidt hammer rebound number (Rn) are not practically reliable enough as in most of the cases one formula is used for all types of rocks, although the density of rocks is introduced to the formulas in some of these cases. On the other hand, if one specific relationship between hammer rebound number and unconfined compressive strength is introduced for one type of rock, the equation will yield a much higher coefficient of correlation. During a research program supported by The Shahrood University of Technology, Iran, a third type of approach was considered. The study aimed to express the relationship between Schmidt rebound number and unconfined compressive strength of rock mass under a particular geological circumstances. As an example, in this study, the situation selected was the immediate roof rock of coal seams at Tazareh Colliery, Shahrood, Iran. In order to determine the Schmidt number and the unconfined compressive strength, a significant number of samples were selected and tested both in-situ and in the laboratory and a new relationship was introduced. The equation can be used to predict UCS of the roof rock in coal extracting areas at this colliery by performing simple in-situ Schmidt hammer tests.

INTRODUCTION

Unconfined compressive strength (UCS) of the rocks plays an important role in many underground and surface rock engineering projects. Determination of the UCS, in theory, is a simple procedure but, in practice, it is among the expensive and time consuming tests which calls for the transportation of the rock to the laboratory, sample preparation based on the existing standards and conducting the tests by using compressive hydraulic jacks.

At these circumstances the application of other simple and low cost methods to carry out the above task with acceptable reliability and accuracy will be important. Among these methods is the application of Schmidt hammer which can be used both in the laboratory and in the field.

As known, the Schmidt hammer has been used worldwide as an apparatus for a quick rock strength assessment due to its portability, easiness in use, rapidity, low cost and its non-destructive procedure of application (Isik, 2002).

During a research work conducted at The Shahrood University of Technology, application of Schmidt hardness in estimating the mechanical properties of rocks, particularly the unconfined compressive strength, under determined geological circumstances was investigated. This paper explains the methodology, test procedures both in the field and the laboratory and analysis and the interpretation of the results.

In addition to the tests carried out in-situ, immediate roof rock samples, predominantly including fine grained sandstone, siltstone and shale have been collected from various locations at Tazareh colliery and tested. The tests included the determination of Schmidt hammer rebound number (Rn) and unconfined compressive strength (UCS). Obtained data were correlated and regression equations were established between Schmidt hammer rebound hardness and unconfined compressive strength, presenting an acceptable coefficients of correlation. It was concluded that there is a possibility of estimating unconfined compressive strength of immediate roof rock, from the Schmidt hammer rebound number by using the obtained equation.

However, the equation must be used only for the hangingwall rock of the Tazareh colliery for estimation of the UCS. In practice by using the Schmidt hammer rebound number obtained from the field or laboratory, in any convenient location at Tazareh colliery, unconfined compressive strength of the roof rock in the location can be estimated with a reasonable accuracy.¹

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³M.Sc. Graduate, Shahrood University of Technology, Shahrood, Iran
SCHMIDT HAMMER

The Schmidt hammer has been used for testing the quality of concretes and rocks. Schmidt hammer models are designed in different levels of impact energy, but the types L and N are commonly adopted for rock property determinations. The type L has an impact energy of 0.735 Nm which is only one third that of the type N (Kahraman, 2001).

Figure 1 shows the details of an L type Schmidt hammer (Torabi, 2005). To perform a test the device is positioned normal to the rock surface and the plunger (13) is pressed against the rock during which the reset spring (1) is pressed and the impact spring (6) is extended. At the end of the course, hammer holding lever (3) contacts the calibration screw (7) and consequently by the rotational movement of the hammer holding lever (3), the hammer is released and after sliding along the plunger neck (11) hits the impact surface of the plunger (12). Based on the hardness of the rock surface onto which the plunger is pressed, the hammer rebounds and the amount of rebound is indicated by the number indicator (10) which now is moved upwards along with the rebound movement of the hammer.

PREVIOUS WORKS

In study of the relationship between the Schmidt number and the UCS, numerous research works have been carried out by others, the notable ones include:

1. Deere and Miller in 1966 (Kahraman, 2001) tested rock cores of the diameter of 55 mm obtained from 28 regions. 48 tests were conducted on each sample. The best fit for the relationship was as follows;

   \[ q_u = 6.9 \times 10^{[0.16 + 0.0087(R_n, \rho)]} \]

   Where \( q_u \) is the uniaxial compressive strength in MPa, \( R_n \) is the Schmidt number and \( \rho \) is the density in g/cm³.

2. Aufmuth in 1973 (Kahraman, 2001) conducted tests on about 800 rock core samples representing 168 geological formations and 25 rock types, The following formula was introduced:

   \[ q_u = 6.9 \times 10^{[1.348 \log(R_n, \rho) - 1.325]} \]

3. Beverly et al in 1979 (Kahraman, 2001) pursuing the Deere and Miller’s attempt, collected samples from another 20 regions and by combining data, introduced the following formula:

   \[ q_u = 12.74e^{[0.0185(R_n, \rho)]} \]

4. Haramy and DeMarco in 1985 (Kahraman, 2001) using Schmidt tests on large sized coal blocks from 10 sites introduced the following formula:

   \[ q_u = 0.094R_n - 0.383 \]

5. Cargill and Shakoor in 1990 (Kahraman, 2001) conducted tests on NX sized rock cores of sandstone and carbonates produced the following equation:

   \[ \ln q_u = 4.3 \times 10^{-2}(R_u, \rho_d) + 1.2 \]
   \[ \ln q_u = 1.8 \times 10^{-2}(R_u, \rho_d) + 2.9 \]

   Where \( \rho_d \) is the dry density of the rock.
TESTS PROCEDURE

Field work was carried out in the Seam P10 at Tazareh Colliery in panels no. 1 and 2 and the roadways containing the roof rock of the seam.

The tests included the application of an L type Schmidt hammer to assess the hardness of the hangingwall rock in as many points as practicable in the stopes as well as in the roadways. In each point an about 20cm by 20cm surface of the rock was prepared by peeling the remaining coal and cleaning the area and performing about 25 tests on each area. Among the numbers obtained, five small amounts were discarded and the mean value of the rest was considered as the Schmidt number for that point. This method of performing Schmidt test was a compromise to the ISRM suggested method (Brown, 1981) where it is argued that the method suffers from some shortcomings due to very selective nature of the procedure (Goktan, 1993). In ISRM suggested method ten higher numbers are selected from twenty tests in the selected area. The applied method in this research work was persistent.

To accomplish the laboratory tests, samples from about thirty points were collected and moved to the laboratory where near cubic shaped samples were prepared of the dimensions of at least 20 cm. After stabilizing the prepared sample on a concrete basement, the same as the procedure followed during the field work and using the same L type hammer, Schmidt tests were performed. In practice, 25 separate points in the surface of the rock specimen were tested and the mean value of the 20 higher values was calculated.

The second phase of the laboratory work consisted of the preparation of the NX sized cores of the rock samples corresponding to the Schmidt tests and conducting direct uniaxial compressive strength tests using a pressure jack (1500 KN, CONTROLS) based on the ISRM standards. Three to five tests were conducted on each specimen.

DATA ANALYSIS

Data from the field and the laboratory were close enough to be used alternatively. Consequently, the results from the laboratory were used to perform the analysis. Previous research (Kahraman, 2002) shows that the correlation between the field and the laboratory data is normally in an acceptable range particularly when the ISRM method is used to conduct the tests.

The data from Schmidt tests and corresponding direct uniaxial compressive strength tests was plotted and best fit was determined as shown in Figure 2.
Tazareh Hangingwall Uniaxial Compressive Strength (UCS)

The best fit to the relationship is as equation (1):

$$UCS = 4.1077R_n - 61.96$$  \hspace{1cm} (1)

The correlation coefficient of the relationship, $R^2$, being in the order of 0.8, indicates that the formula can be acceptable only in the preliminary stage of assessment and for more detailed investigations additional measures should be applied. On the other hand, high dispersion of the data in lower Schmidt numbers (below around 35) as shown in Figure 2, indicates that this method is not reliable within this range. This range corresponds to the Shale and part of Siltstone in the roof rock.

Attention should be paid to the fact that firstly this relationship is unique for this geological situation and secondly in the case of relatively high dispersion of rock types in a specific geological situation, it might be more advisable to use the existing relationships.
CONCLUSIONS AND SUGGESTIONS

Unconfined compressive strength of rocks plays a significant role in rock engineering projects. As a simple tool for quick UCS assessment, Schmidt hammer has been used worldwide. In order to calculate the UCS using the results of Schmidt tests different types of formulas were introduced by researchers.

Review of the literature showed that the early relationships, where one formula covered all types of rocks, were not reliable. The relationships in which the density of the rock was introduced yielded more acceptable results. On the other hand, the formulas which were used for a particular type of rock, yielded more reliable results. In this research work, however, a new approach was considered where a specific geological situation, in this case the hangingwall rock of the Tazareh colliery was selected and a relationship was developed. The resulting formula can be used to assess the UCS of the hangingwall of this colliery by performing simple in-situ Schmidt tests.

It is advisable that such a procedure be followed in considering any colliery to study and a unique relationship between the Schmidt number and UCS be developed. The obtained relationship can be used as a quick reference to suggest a preliminary value for UCS at any point in the colliery during the mine life. Furthermore, as another outcome of this study, in addition to the collieries’ roof strata, other specific geological or geotechnical situations can be selected and tested. In this context it is presumed that the geological agents acting on the formation has imposed some common characteristics on the rock types forming the formation rendering it homogeneous in response to the hardness tests. However, this is practicable only if the dispersion of the rock types in the formation is not high causing the correlation coefficient to fall into an unacceptable range, otherwise the existing relationships introduced for different types of the rocks will be more acceptable.

ACKNOWLEDGMENT

The authors wish to acknowledge the cooperation of the Tazareh Colliery management in providing facilities and access to the site and information. Also the research department of the Shahrood University of Technology is acknowledged for funding this research program.

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INNOVATIVE CFD MODELLING TO IMPROVE DUST CONTROL IN LONGWALLS

Ting Ren and Rao Balusu

ABSTRACT: Reducing dust exposure of operators on longwall faces remains a challenging issue for mine managements. Most of the Australian mines are adopting uni-di cutting method to reduce operators dust exposure levels. Even in this uni-di cutting mode, the dust roll-up towards the walkway area is very high in most cases and is resulting in high dust exposure levels for shearer and chock operators.

With the support of ACARP, CSIRO has been undertaking several research projects (C12025, C13019 and C14036) based upon CFD modelling to improve the understanding of dust flow patterns around the longwall shearer and walkway under different operating conditions, and the study of a range of dust control options/concepts for reducing operators dust exposure levels. During these simulation studies, a shearer scrubber system has shown to be capable of significantly modifying the airflow patterns around the maingate cutting drum and reducing dust roll-up towards the walkway area.

INTRODUCTION

The behaviour of respirable dust in a longwall face is a complex process because of the nature of longwall operations. The generation, dispersion and transport of airborne dust is governed mainly by the spatial velocity and the movement pattern of the ventilation air. To understand the dust behaviour in a complex longwall mining environment and to evaluate the effectiveness of various dust control techniques, numerical modelling has become a necessity to supplement laboratory experiments and field studies.

CFD codes have been successfully used in South Africa and Australia in areas such as simulation of airflow patterns around coal cutting machines in development headings and longwall faces (Sullivan & Van Heerden 1993, Balusu et al. 1993). Results of the development heading study were used to investigate the effect of onboard scrubber and ventilation systems and to determine if the addition of a jet fan could minimise some of the negative effects of such systems. CFD models also helped in the investigation of the feasibility of different scrubber intake designs and establishing the most effective location for such intakes.

Recent work in CFD modelling of dust problems in longwalls and development roadways has demonstrated that this technique has major advantages over conventional numerical modelling for an improved understanding of air flow fields and dust behaviour in a three dimensional environment (Balusu et al., 2005). CFD techniques also provide a powerful tool for initial concept testing of new and innovative ideas for dust control.

This paper describes the development of 3D CFD models at CSIRO to investigate the airflow behaviour and the use of various controls on respirable dust dispersion on the longwall faces.

LONGWALL CFD MODELS

Three dimensional CFD models have been developed to investigate the airflow behaviour and respirable dust dispersion patterns in longwall faces mining thin, medium and thick seams. These models consist of a section of the full-scale coal face and the maingate, and embody the major longwall components such as chocks, shearer, spill plate, BSL/crusher and conveyor. Figure 1 shows the computational grid of the CFD model of the longwall shearer.

Base model simulations were carried out with a variety of intake airflow rates, ranging from 30m$^3$/s to 110m$^3$/s, and were calibrated and validated against field airflow velocity data obtained from three Australian underground coal mines.

In these simulations, a cluster of respirable dust particles (particle sizes between 1~10 µm) were ‘released’ at various locations, mostly from the face spalling area and at a distance ahead of the cutting drum. The dispersion of particles in the face airflow was tracked by using the stochastic tracking (random walk) model. The function provides a powerful means of visualising the dust dispersion patterns and the impact of dust control methods such as the use of shearer scrubbers on dust capture and diversion. Figure 2 illustrates the tracking of respirable dust particle in the CFD modelling around the longwall shearer.

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The validated longwall CFD models were used to investigate the effect of mining and dust control options on face air and respirable dust flow patterns. These include face ventilation rate, drum cutting sequence, the use of sprays/venturi's and Shearer Clearer system, air curtains and shearer dust scrubbers.

Figure 1 - Computational mesh of the CFD model – the longwall shearer

Figure 2 - Respirable dust particle tracking in CFD modelling around the longwall shearer
CFD MODELLING OF VENTILATION AND DRUM CUTTING SEQUENCE

CFD results indicate that the dispersion of respirable dust on a longwall face is largely dependent on the location of dust generation and airflow patterns in that area. The dust dispersion from the cutting drum and face spalling is slightly more extensive on thick seam longwall faces with low airflow rate of around 35 m$^3$/s compared with higher airflow rates of around 55 m$^3$/s and 80 m$^3$/s. With face ventilation increased to 55 m$^3$/s, most of the dust particles from face spalling would be confined to the face/shearer area and away from the working zone of face operators.

With shearer cutting from MG to TG, instead of the standard TG to MG, it would reduce air diversion and the subsequent dust migration towards the walkway area. Cutting from TG to MG with a reversed drum (MG drum cutting at floor or mid-seam level, instead of at roof level) would reduce dust migration towards the walkway significantly, compared with dust migration in standard cutting mode. However, it is to be noted that some operational issues may restrict the application of these modified cutting sequences.

CFD MODELLING OF SPRAYS/VENTURI’S AND SHEARER CLEARER SYSTEM

Modelling results demonstrate that sprays/venturi’s mounted on the shearer body operating at a low flow rate of 0.1m$^3$/s would have only a limited impact near the upwind cutting drum area. However venturis operating at higher flow rates of about 0.5m$^3$/s would have a significant effect on airflow and the behaviour of respirable dust at both the cutting drums. In both cases, the sprays/venturis facing the downwind TG drum on the shearer body tend to pull the dust cloud towards the face. However, sprays/venturis directed at the upwind cutting drum significantly increases dust dispersion towards the walkway area.

As shown in Figure 3, a Shearer Clearer operating at 0.25 m$^3$/s flow rate would help reduce the dispersion of dust particles towards the face operators by inducing the dust cloud towards the face area, and at a higher flow rate of 0.5m$^3$/s, the performance of the Shearer Clearer system can be improved significantly. However, the area of influence of a ‘Shearer Clearer’ is very limited in thick seam environments and seems to have only marginal effect in reducing dust migration towards the walkway area. Shearer Clearer operating at higher flow rates of around 0.5m$^3$/s in a less than optimal direction can cause excessive turbulence, and may result in a significant dust migration towards the walkway area.

Figure 3 - Effect of shearer clearer on respirable dust flow patterns – plan view (dust released near the coal spalling area)
CFD MODELLING OF AIR CURTAINS

CFD modelling results indicate that standard square air curtains along the shearer would only have limited impact on diverting the mainstream airflow in general and therefore marginal effect on separating the respirable dust particles from the shearer operators. Low height curtains installed over the shearer body in parallel to airflow seems to be negligible in reducing face operators’ dust exposure levels. Curtain installed near the shearer at an angle to the airflow seems to substantially alter the air and dust flow patterns and develop some recirculation zones around the curtain. The correct combination and orientation of curtains near the stage loader would substantially reduce the face operators’ exposure to intake dust levels.

Airfoil curtains appear to have good potential in diverting airflows and subsequently dust particles away from shearer operator. These airfoil curtains, as indicated by CFD modelling in Figure 4, when properly aligned and attached to the shearer or ahead of the shearer, have the potential to modify the airflow patterns significantly in the vicinity of the shearer by diverting the air stream towards the face, thus helping the confinement and separation of respirable dust particles away from the walkway of the shearer operator.

CFD MODELLING OF SHEARER SCRUBBER SYSTEMS

To prevent dust from becoming airborne, one of the several successful control measures in longwall faces is the use of dust scrubbers which cleanse the air and help to prevent the contaminated air from reaching areas used by mine personnel. Scrubbers have proven to be highly successful in reducing airborne dust at BSL and other conveyor transfer points. However, shearer-mounted scrubbers have not been used successfully on longwall faces. The capacity of a shearer dust scrubber is important in terms of dust capture efficiency and diversion impact on face airflow patterns.

Modelling results indicate that shearer scrubbers with flow capacity up to 4 m$^3$/s will have only a marginal effect on respirable dust flow patterns near the shearer, and the capture and diversion of the respirable particles would be significantly improved with a shearer scrubber operating above 8m$^3$/s.

The scrubber capacity must be appropriate in relation to the prevailing face airflow rates (velocities) around the shearer. Scrubbers with a capacity around 8m$^3$/s would have a good impact on dust capture and dust flow patterns at moderate face ventilation of around 55 m$^3$/s. However, the capture efficiency of the scrubber (at 8m$^3$/s) would be significantly compromised for longwall face with a high ventilation flow rate (above 80m$^3$/s). In practice, a 10m$^3$/s scrubber might be the maximum capacity that can be installed on the shearer due to restrictions on the longwall faces. With this capacity and a moderate face ventilation rate (less than 60m$^3$/s), the scrubber should be sufficient to capture a good portion of the dust particles and in the meantime modify the flow patterns near the shearer.

As shown in Figure 5, both the location of scrubber inlet and outlet are important in the design of an effective dust scrubber system. Scrubber inlet located on the edge of the shearer body facing the ventilation direction and close to the dust generating sources offers an improved advantage in capturing dust particles from both the spalling area and the roof ahead of the shearer. This position is particularly effective for capturing dust particles dropping from the roof and chock movements ahead of the shearer near the spalling area.

Scrubber outlet discharge angled slightly towards the face would help the confinement of dust particles to the face and the overall diversion of dust clouds away from the walkway; whilst if the outlet discharge is set at an angle against the general airflow stream, it would deflect a high portion of the escaped dust particles towards the walkway and downwards along the face, even if the scrubber has a high dust capture efficiency. Reduced scrubber outlet airflow velocity and turbulence would also help reduce dust particles roll-up into the operators' walkway on downwind side of the shearer.

(a) Plan view of dust particle capture and flow patterns - scrubber discharge at 10˚ towards the face
(b) 3D view of dust particle capture and flow patterns - scrubber discharge at 10˚ towards the face

Figure 5 - Dust particle capture and flow patterns with the new shearer scrubber capacity at 10m3/s – scrubber discharge 10˚ towards the face

In general, the modelling results indicate that total dust capture is not feasible for a dust scrubber attached to the shearer. The design of the scrubber system should be aimed to capture a proportion of the dust particles and modify the face flow patterns to divert respirable dust clouds from the shearer operator’s position in the walkway area. This can be achieved by correctly positioning the scrubber inlet, discharge direction and matching the scrubber capacity with the face ventilation rate.

DEVELOPMENT OF A NEW SHEARER SCRUBBER FOR FIELD TRIAL

A new dust scrubber has been designed as a key part of the ACARP project C14036. The design of the new scrubber has incorporated the CFD modelling findings, including the desired scrubber capacity, the inlet locations and the airflow discharge direction from the elutriator. The manufacture of the shearer scrubber has been completed and is ready for trialling at a suitable mine site to evaluate the system’s airflow pattern modifying capability near the leading cutting drum. These initial field trials will include evaluation of the scrubber system robustness, its interaction with other face cutting or shearer maintenance activities, its noise levels and the effect of curtains on operator’s vision of the face cutting operations. The scrubber system’s airflow modifying capability will be investigated in detail by taking a number of field measurements of airflow across a number of sections along the face. This will ensure trial results will be measured against CFD modelling results.

CONCLUSIONS

CFD modelling simulations proved to be an invaluable tool in understanding air and dust flow patterns on a longwall face and in studying the effect of various mining and operational parameters on dust dispersion and flow patterns.

Modelling results showed that cutting from MG to TG direction would reduce air diversion and the subsequent dust migration towards the walkway area; Simulations with modified cutting sequence in TG to MG direction with reversed drum showed that respirable dust migration towards the walkway reduces significantly, compared with dust migration in standard cutting mode. However, other operational issues may restrict the application of these techniques in some mines.
Sprays or venturis directed at the upwind cutting drum significantly increases dust dispersion towards the walkway area. Simulation results indicate that the area of influence of a ‘Shearer Clearer’ is very limited in thick seam environments and seems to have only a marginal effect in reducing the face operators’ dust exposure levels.

A new dust scrubber has been designed and manufactured as a key part of the ACARP project. The design of the new scrubber has incorporated the CFD findings, including the desired scrubber capacity, the inlet locations and the airflow discharge direction from the elutriator. The manufacture of the shearer scrubber has been completed and is ready for trialling at a suitable mine site.

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UNDERGROUND ATMOSPHERE REAL TIME PERSONAL
RESPIRABLE DUST AND DIESEL PARTICULATE MATTER
DIRECT MONITORING

Stewart Gillies and Hsin Wei Wu

ABSTRACT: An overview is given of two new developments in mine atmospheric monitoring. A new personal dust monitor (PDM) that gives realtime respirable dust readings is discussed. The unit is mounted within the miner’s cap lamp battery and internally measures the true particle mass of dust collected on its filter. Samples are available for later mineralogical analysis and results do not exhibit the same sensitivity to water spray as optically based measurement approaches. The technique achieves microgram-level mass resolution even in the hostile mine environment and reports dust loading data on a continuous basis. The monitor has been evaluated under an Australian Coal Association Research Program (ACARP) grant and is being adopted for statutory mine respirable dust determinations in the US. It has particular application for determining high source locations and efficiency of engineering means of suppression and other approaches to handling the problem.

It has been recognised that the PDM’s unique measurement approach has application to allow real time atmospheric Diesel Particulate Matter (DPM) monitoring. The industry has no real time atmospheric DPM monitor at present. Recent surveys in New South Wales and Queensland continue to show significant numbers of miners continue to face full shift DPM exposures in excess of internationally accepted levels. Real time DPM monitoring will allow the industry to pin-point high exposure zones where a number of trucks and other vehicles work or in areas of poor ventilation. Pinpointing of high DPM concentration zones will allow efficient modification of work practices to reduce underground miners exposure. Approaches to design of Tag Boards are also discussed. Some outcomes from mine tests with both these new instruments are discussed.

INTRODUCTION

Mine ventilation is a critical aspect of all underground mines. Mining technological developments and mining environment challenges are necessitating new approaches. This paper in particular examines two areas of new development.

The coal industry is vigorous and expanding and driven by high prices and export demand. The push is unrelenting for increased production rates particularly from longwall production. Faces quantities and velocities continue to increase in raised gas, dust and heat level environments.

Many of our mines face high seam gas levels in conjunction with high propensity to spontaneous combustion. There will continue to be better and more innovative approaches to gas drainage. Atmospheric inertisation was first introduced as a tool to fight fires. It is now accepted as a component of the production cycle in some mines.

The network in many modern mines changes daily as stopes or development breaks through. Maintaining an understanding of the ventilation network is a challenge. Improved use of real time monitoring and control may, in time, allow mines to optimise this situation. Instrumentation developments are allowing improved realtime monitoring of ventilation parameters and particularly gases, respirable dust and airflow. Understanding fires, simulation of fires and training the workforce will continue as a priority area.

Ventilation expenditure receives priority when it directly affects production. It is up to the ventilation practitioner to point out the real cost of the ventilation system to the overall mine capital and operating costs. Ventilation costs are not just fan electricity costs and ventilation control device budgets as some may see it. The layout of a mine is largely dictated by ventilation requirements. The provision of a pleasant and comfortable work environment returns increased miner productivity.

Many of the new developments will be contributed to by research activities. ACARP has been outstandingly successful in supporting focusing research efforts to productive coal industry benefit. The 5 cents per export tonne levy has been leveraged by additional co-sponsoring by operating companies, universities and others. Grants from this source carry prestige and it is hoped the real value of the program will continue.

Various mining industry accidents or disasters have led to, or reinforced, a revolution in thinking in many areas of management of the industry. Regulations are less prescriptive and now demand risk assessment incorporating

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international best practice. Australia is at the international forefront here. There is a much grater emphasis on training at all levels. Much of the industry is actually or effectively long distance commute (such as Fly In Fly Out). It is beyond the scope here to cover the issues of joint management, longer work shift hours and so on that this presents to the management of ventilation. There is more use of consultants than ever before, a situation that again presents many issues.

Vehicles for publication of ventilation innovation for dissemination to the wider industry community are becoming fewer. It is the specialist conferences that have become the main archival repository of our thinking and innovations for reference in the future. The two areas of new development discussed within this paper have been supported by industry research grants from in particular the ACARP with substantial input from the United States agency, the National Institute of Occupational Health and Safety (NIOSH). They are stories in practical application and have received considerable additional industry financial support, mine site testing and evaluation assistance.

MONITORING OF RESPIRABLE DUST

A new PDM for respirable dust developed by the company Rupprecht and Patashnick (now Thermo Fisher Scientific) in the US under a project funded by the NIOSH has generated promising results in underground coal mine testing performed in the US recently (Volkwein et al, 2004a and 2004b). Results from an ACARP funded study undertaken to evaluate this new realtime dust monitor for personal respirable dust evaluation particularly in engineering studies have been described by Gillies, 2005 and Gillies and Wu, 2006.

This paper describes some results from mine studies that have been undertaken using the real-time PDM. The technology that forms the heart of the PDM, the TEOM® system, is unique in its ability to collect suspended particles on a filter while simultaneously determining the accumulated mass. The monitor internally measures the true particle mass collected on its filter and results do not exhibit the same sensitivity to water spray as optically based measurement approaches. The technique reports dust loading data on a continuous basis and miners and mine operators have the ability to view short term dust levels. It is believed to be the first personal dust monitor instrument that reliably delivers a near-real-time reading.

The instrument has potential to be used as an engineering tool to evaluate the effectiveness of dust control strategies. Being a personal dust monitor, the instrument measures the airborne dust from the breathing zone of any worker. It provides an improved understanding of the potential hazards faced by workers. The PDM will highlight high dust situations and allow the situation to be corrected. The underground workplace in both continuous miner and longwall face environments has varying respirable dust conditions due to aspects such as ventilation conditions and air velocity, shearer activity and design, chock movement, armoured face conveyor movement, mining position, face time of individual personnel, outbye conditions and dust levels in intake air and measurement instrument behaviour. A study has evaluated the instrument as an engineering tool that can assess the effectiveness of a single change to improve dust levels in sufficiently short a time that other aspects have not changed.

The PDM is a respirable dust sampler and a gravimetric equivalent analysis instrument that is part of a belt-worn mine cap lamp battery. The main components of the device include a cap lamp and sample inlet located on the end of an umbilical cable, a belt-mounted enclosure containing the respirable dust cyclone, sampling, and mass measurement system, and a charging and communication module used to transmit data between the monitor and a PC while charging its lithium ion batteries for the next shift. Figure 1 illustrates the unit.

The current US Federal congressional legislative program includes responses to strengthen mine emergency response plans and the Mine Safety and Health Administration's ability to investigate accidents, enforce health and safety regulations, strengthen rescue, recovery and accident investigation practices and update the 37 year old respirable dust standard that is not effectively preventing today's miners from developing black lung disease. Part of this move may require miners to be equipped with the new PDMs developed and certified by NIOSH and authorise miners to adjust their activities to avoid respirable dust overexposure.

Tests for underground evaluation exercises were undertaken at a development face to monitor the dust exposure levels of various equipment operators. The PDM units can give variable time period rolling averages of dust concentration and for engineering evaluation purposes it is better to use shorter time rolling average dust concentration data (such as 55 mins) as the quicker response to monitored changes shows more significant dust concentration variations. As shown in Figure 2 a development face was monitored. Two PDMs were used with one worn by the Continuous Miner (CM) operator and one by the bolter. The CM operator was using a remote control unit and stood on the right of the heading. The bolter was using the left hand machine mounted unit. Ventilation to the face area was good and ducting was extended approximately every 25 minutes.
The exposure levels experienced by the CM operator who was standing very close to the open end of the exhausting ducting and so was in the best face area ventilation stream were consistently lower than those recorded by the bolter. During the period from 17:20 the CM holed through to a previously mined cut through. It is clear that the detrimental change caused in face ventilation from the hole through overwhelmed any change in relative exposure recorded by the two face crews because of the geographic positioning.

The longwall panel has a number of potential dust sources. A detailed survey can assist in evaluating the contribution of each component source, show the contribution from a number of major sources and the cumulative dust level faced by a miner at different points throughout the panel. In undertaking Longwall studies it is important to maintain consistency with measurement conditions along the face activities. Figure 3 indicates studies undertaken over the majority of a shift with two PDM units. The shearer position data was downloaded from the mine monitoring system. A cutting sequence
generally took 90 to 120 minutes. It can be seen in the figure that 4.5 complete cutting cycles occurred across the 9.5 hour study time period with good regularity. One afternoon period of almost two hours was lost to a breakdown.

Figure 3 - LW Face Dust Surveys Shearer Position and dust monitored points and Levels

Figure 4 examines variation of dust make with shearer advance rates in tests in the same mine. Two MG to TG cuts were examined; one taking over 31 minutes for the cut and the other only taking 25 minutes. It is clear that although there is virtually the same dust make in the two cuts at the same operator position (inbye of the TG shearer drum) the dust exposure of average 1.22 mg/m$^3$ for the faster cut is greater than for the slower at 0.91 mg/m$^3$.

Figure 4 - Variation of dust make with shearer advance rates

One of the LW faces tested advances the first five MG chocks during the TG to MG cutting sequence due to roof condition. Large amount of dust during this chock advance are generated and face operators are exposed to this dust as the cloud passes along the face. Figure 5 shows for the shearer MG operator position comparisons of dust generated by MG chocks (1 to 5) advance sequences (highlighted by hatching) and LW full cutting cycle face dust measured for two consecutive shearer cutting cycles. Almost half (48.2 and 49.8%) of this particular LW face dust
exposure generated during the cutting sequence is comes from MG chock advance dust at this particular LW mine. In tackling respirable dust reduction lessening, removing and/or isolating this chock dust source is warranted.

**Longwall Face PDM Measurements**

(5 minutes rolling average)

<table>
<thead>
<tr>
<th>Time</th>
<th>Respirable Dust Conc (mg/m³)</th>
<th>MG Chocks 1-5 Advance Dust Generated</th>
<th>DPM MG Shearer Operator</th>
<th>Shearer Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>21:30-22:30</td>
<td>2.80 mg/m³</td>
<td>104.9 g</td>
<td>20.0 mg/m³</td>
<td></td>
</tr>
<tr>
<td>22:30-23:00</td>
<td>1.16 mg/m³</td>
<td>217.5 g</td>
<td>21.0 mg/m³</td>
<td></td>
</tr>
<tr>
<td>23:00-23:30</td>
<td>2.85 mg/m³</td>
<td>106.6 g</td>
<td>21.5 mg/m³</td>
<td></td>
</tr>
<tr>
<td>23:30-0:00</td>
<td>1.14 mg/m³</td>
<td>214.0 g</td>
<td>21.0 mg/m³</td>
<td></td>
</tr>
</tbody>
</table>

Ratio of MG Chock and Total Dust generated per sheaver cycle

- 21:39-22:39 = 104.9 / 217.5 = 48.2%
- 22:40-23:36 = 106.6 / 214.0 = 49.8%

**Figure 5 - MG chocks 1-5 advance dust compared with total LW face dust at shearer MG operator position**

Based on the tests conducted, it is concluded that the PDM has demonstrated its potential use as an engineering tool to locate and assess various sources of dust during normal mining operations. The principles and concepts used to identify and fix some of the higher dust levels are generally common sense. However, to make the most effective use of this information, training and experience in using this type of technology will be very important. Experience with the data from the unit will help miners gain confidence to use the information to maintain reduced or safe dust levels during mining.

**MONITORING OF DIESEL PARTICULATE MATTER**

DPM issues are very high profile currently in both Australian coal and metalliferous mines. Mine atmosphere measurements of DPM in Australian mines have only been measured systematically since mid 2000s. Early atmospheric readings have been taken on a shift average basis using SKC sampling units. The SKC is derived from a US NIOSH design and gives readings in the surrogate Total Carbon (TC) or Elemental Carbon (EC) units after laboratory analysis procedures have been completed.

- DPM = TC + inorganics = EC + organic carbon (OC) + inorganics
- TC in mine testing is consistently over 80% of DPM (Volkwein 2006).

Some DPM regulatory guidelines are starting to emerge in Australia. However no prescriptive mining regulations are in force internationally although the US metalliferous mining industry is to face mine atmosphere DPM regulations from April 2008. Australian states are generally moving to acknowledge US April 2008 final metal mine regulation limits of 0.2 mg/m³ submicron particulate matter, 0.16 mg/m³ total carbon particulate and 0.1 mg/m³ elemental carbon particulate.

The real time DPM monitor is being developed on the base of the successful PDM unit. A description is given of an underground series of tests undertaken to establish the robustness and reliability of the new approach. Thermo Fisher Scientific has undertaken structural changes to the PDM to convert it to a DPM real time monitoring underground instrument, the D-PDM. The Pennsylvania Pittsburgh Research Laboratories of NIOSH (the group that originally contracted for the PDM development) has undertaken laboratory “calibration or verification” testing. They have an accredited diesel exhaust laboratory and international expertise in this area. The D-PDM directly reports levels of mine atmosphere DPM in mg/m³ from real time readings. It can be placed in the working place or in a mine vehicle and when design is finalised will be able to be worn by a person.

The D-PDM instrument is currently at a prototype stage and as with all new technologies will need industry acceptance and support to reach its full potential.

A phase of Australian mine robustness and engineering testing has been successfully undertaken in four mines to ensure the instrument can effectively assist mine management to handle this health issue. Tests described have been undertaken at points of expected high atmospheric DPM such as vehicles movements, during Longwall face
moves and in an exercise in Tag board design. The outcome of the project gives the industry access to an enhanced tool for understanding the mine atmosphere in the presence of DPM.

The Mine 1 tests were undertaken in working sections with use of diesel powered Ram cars. The results from these limited tests qualitatively indicated that D-PDM did respond to observed diesel activity in fairly low concentration ranges. It was found that 10 minute rolling averaging periods appear to allow a balance between ability to recognise individual diesel source vehicle movements and measurement accuracy. Some readings were taken with instruments mounted on a vehicle with positive results.

Mine 2 testing exercises monitored various ventilation arrangements of a longwall face move during chock transport to the installation roadway. It was straightforward to analyse results for arrival and departure times of diesel machines at the face. Interpretation could be made on whether the machine travelled down gate roads either with a speed faster than the air velocity (and so with high exhaust concentrations trailing) or with a speed slower that the air velocity (and so with high exhaust concentrations in advance).

The longwall ventilation arrangement for one set of tests is shown in Figure 6. The positions of the D-PDM monitors #106 and #108 are shown; #106 in the face installation road and #108 in a cut through ventilating the face. On this test day loaded chock carriers travelled in along the main gate (MG) and out through tail gate (TG). About 50 m³/s ventilation was measured in the MG and about 35 m³/s in the TG. There was a raise borehole upcasting some air.

![Figure 6 - Longwall ventilation-chock carriers travel in on MG and out on TG](image)

Four chock carriers were available and a total of 10 chocks were moved. Results from monitor #108 as shown in Figure 7 clearly demonstrated the ability of the D-PDM units to detect variations of DPM levels in the atmosphere as the Chock carriers travel in from MG and out from TG of the LW face. Significant submicron DPM readings were recorded due to the large number (10) of chocks that were transported during the shift. Levels of DPM recorded in the second half of the shift were higher. The condition of the back road had become poor and some chock carriers were slower and having difficulty travelling through.

![Figure 7 - Observations on results at monitor 108 fixed location](image)
Figure 8 examined one three hour period with particular interest in recording of D-PDM readings as compared to Heading air velocity chock carrier vehicle speed. Close examination of results from #108 monitoring the DPM downstream of the MG and back road showed that when the chock carriers travel in from the MG in three cases they arrived at the TG end of the face in advance of the peak level of the DPM cloud. This indicated that the carriers were generally travelling at higher average speed than air velocity. However Carrier #1112 arrived slightly later indicating slower machine travel speed than air velocity. The time difference and also the peak concentration depends on the air velocity and chock carriers’ travel speeds. In theory if the chock carrier travels at the same speed as air velocity the peak concentration will be extremely high and the carrier will arrive at the same time as the concentration peak.

Mine 3 exercises monitored various ventilation arrangements of longwall face move during chock transport to the installation roadway. Figure 9 shows Longwall ventilation arrangement for tests and the positions of the D-PDM monitors #106 and #108 during the tests. On this test day loaded chock carriers travelled in and out through the TG. About 28 m³/s ventilation was measured in the MG and about 39 m³/s in the TG. There was a back borehole downcasting about 11 m³/s. Three chock carriers were available and a total of four chocks were moved.
significant DPM levels added along the Longwall face due to the installation activities of chocks by “shunting mules” or LHDs. The largest source was from chock carriers that carried individual chocks along the length of the TG to reach the face.

![DPM Survey at LW Move Activity Day 2 2007](image)

Figure 10 - DPM make from LW face activity, #110 compared with DPM make from both face and TG transport activities, #106

<table>
<thead>
<tr>
<th>Location</th>
<th>Sources, μg/s</th>
<th>%</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>MG, C &amp; D Hdgs</td>
<td>3.03</td>
<td>18.6</td>
<td>Mains air at MG panel entrance</td>
</tr>
<tr>
<td>Borehole</td>
<td>0.00</td>
<td>0.0</td>
<td>Situated at the back of LW panel, fresh air</td>
</tr>
<tr>
<td>LW Face</td>
<td>4.77</td>
<td>29.2</td>
<td>Shunting Mule or LHDs</td>
</tr>
<tr>
<td>TG D Hdg</td>
<td>6.96</td>
<td>42.6</td>
<td>Chock carriers travel way</td>
</tr>
<tr>
<td>TG C Hdg</td>
<td>0.00</td>
<td>0.0</td>
<td>No diesel activity</td>
</tr>
<tr>
<td>Leaksages</td>
<td>1.57</td>
<td>9.6</td>
<td>Mains air; coffin seal &amp; double doors</td>
</tr>
<tr>
<td><strong>Measured Total</strong></td>
<td><strong>16.32</strong></td>
<td><strong>100.0</strong></td>
<td></td>
</tr>
</tbody>
</table>

As discussed by Dabill (2005) exposure of drivers of diesel vehicle to DPM can be limited by the direction of travel and the ventilation system. For vehicles travelling against the ventilation always try to ensure the engine is trailing the driver. Under these conditions driver exposure to DPM will be low if there are no other vehicle inbye. However, travelling against the ventilation flow with the engine forward can lead to very high driver exposure and where possible this should be avoided or at the very least reduced to as short a time as possible.

It is more difficult to minimise exposure when travelling with the airflow as no matter what speed the vehicle travels the driver is likely to be exposed. It is important for the vehicle not to travel at the same speed as the ventilation air velocity as the vehicle driver will be operating in an ever increasing concentration of diesel exhaust emissions and consequently exposure could be very high. If the vehicle is likely to be travelling faster than the ventilation airflow then have the engine trailing and if the vehicle is slower than the ventilation have it orientated with the engine forward of the driver. By observing these rules exposure to DPM will be kept to a minimum but will not be eliminated altogether. Table 2 demonstrates vehicle speed and ventilation air velocity over a single travel route, Mine 3 TG Heading D, for face chock delivery.

Points that can be established from this data.

- In these specific tests chock carriers travel at higher average speed than air velocity.
- However on poor roads there could be slower machine travel speed than air velocity.
- The time difference and the peak concentration will depend on the air route, whether the air is travelling with or against the carrier direction, the air velocity as a function of the air quantity and chock carriers’ travel speeds.
In theory if the chock carrier travels with the air at the same speed as air velocity the peak concentration around the vehicle could be extremely high.

**Table 2 - Data on chock carrier vehicle speeds and air velocities and machine against air relative velocities**

<table>
<thead>
<tr>
<th>Time</th>
<th>Location</th>
<th>In/Out</th>
<th>Distance m</th>
<th>Time mins</th>
<th>Speed, m/s</th>
<th>Air Vel m/s</th>
<th>Air Travel Time mins</th>
</tr>
</thead>
<tbody>
<tr>
<td>9:53</td>
<td>TG26 2ct</td>
<td>In</td>
<td>3,400</td>
<td>34</td>
<td>1.66</td>
<td>1.29</td>
<td>43.9</td>
</tr>
<tr>
<td>10:27</td>
<td>Face</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10:31</td>
<td>Face</td>
<td>Out</td>
<td>3,400</td>
<td>26</td>
<td>2.18</td>
<td>1.29</td>
<td>43.9</td>
</tr>
<tr>
<td>10:57</td>
<td>TG26 2ct</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Chock Carrier APS 1306**

- Machine Against Air
- Machine/Air Rel Velocity, m/s = 2.95
- Machine With Air
- Machine/Air Rel Velocity, m/s = 0.89

<table>
<thead>
<tr>
<th>Time</th>
<th>Location</th>
<th>In/Out</th>
<th>Distance m</th>
<th>Time mins</th>
<th>Speed, m/s</th>
<th>Air Vel m/s</th>
<th>Air Travel Time mins</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:12</td>
<td>TG26 2ct</td>
<td>In</td>
<td>3,250</td>
<td>28</td>
<td>1.93</td>
<td>1.29</td>
<td>41.9</td>
</tr>
<tr>
<td>10:04</td>
<td>TG26 36ct</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10:05</td>
<td>TG26 36ct</td>
<td>Out</td>
<td>3,250</td>
<td>17</td>
<td>3.18</td>
<td>1.29</td>
<td>41.9</td>
</tr>
<tr>
<td>11:07</td>
<td>TG26 2ct</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Chock Carrier CC 1112**

- Machine Against Air
- Machine/Air Rel Velocity, m/s = 3.22
- Machine With Air
- Machine/Air Rel Velocity, m/s = 1.89

A possible reduction in DPM driver exposure could have been achieved by consideration of the following.

- TG travel route panel air quantity could be increased.
- Alternatively TG air could be re-routed, eg Air into panel up D Heading and return down C Heading.
- Increase in air velocity may result in relative air velocity and vehicle speed being very similar. This is to be avoided if vehicle travels with air as would have happened if vehicles came into the panel up D Heading.
- Best if vehicle travels against airflow direction.
- Best conditions would be achieved if air came into panel up D Heading and returned down C Heading and traffic was in the opposite and drove up C and down D Headings. In this configuration vehicles would always travel against air. If the vehicle exhaust outlet trails the driver then it will pass away from the driver in both directions of travel.

DPM tests were undertaken in Mine 4 to evaluate whether the use of the D-PDM could contribute to the design of a Tag board. Tag boards are relatively new to the mining industry and are currently used in only a small number of mines. Tag boards are used to limit access of diesel vehicles entering a particular ventilation split or mining sections to manage exhaust DPM and gases. Diesel tags or tokens are used to control the number of vehicles entering and so limit level of pollution. Existing Tag board systems are based on historic workshop tailpipe readings and mine plan projections of air quantity availability. A new vehicle to a section is stopped from entering until the acceptability of the current atmosphere as determined by a check as to whether a spare tag position is available is made.

An alternative approach is to invest in underground continuous real time monitoring of exhaust gases, DPM and section air quantity and integrate this information to determine whether an additional vehicle can enter without exceeding diesel token limit. This approach optimises the access of diesel vehicles and replaces the existing manual tag board system based on historic workshop tailpipe readings. This system would allow productivity improvement by detecting dirty engines and permitting the maximum number of vehicles to be in use in a ventilation split based on real exhaust contamination. The basis of the system is to determine whether an additional vehicle can enter without exceeding the section ventilation split DPM or gases limits. Currently the pre-determined “tag” allowance may be excessively stringent for a well maintained vehicle and so vehicles have to wait and waste time until another vehicle leaves the section ventilation split.

A real time monitoring approach puts on an objective basis the process for determining how many vehicles can be in the ventilation circuit of an underground section. Currently systems in place across various mines refer to historic workshop tailpipe readings or manufacturers’ guidelines. A particular vehicle may be determined to require for instance one or two tag positions on the board before entering a section. This approach is pragmatic but does not account for many aspects of engine performance or maintenance status. The real time system could be tied to a mine vehicle tracking system (of which a number of commercial systems are available) to identify individual units. This approach would actually measure the exhaust DPM and CO gas contaminant in the ventilation circuit with a number of vehicles present and determine whether a predetermined limit has been reached before allowing access of additional vehicles through the access or tracking system entry point.
From a brief review of the Australian mining industry it is concluded that there is currently no generally accepted industry approach to Tag Board design. Those that exist have mostly been designed from exhaust gas level considerations. Some are designed from ventilation indices for engine exhaust gas output such as 0.06 m³/s/kW output. Some are designed from OEMs’ published ventilation requirements for exhaust gas outputs for particular engines. Recently some mines have started to take account of engine exhaust DPM from Bosch meter tests (smoke interference) in Workshop tests. SIMTARS (a section of the Queensland Department of Mines and Energy) has been collecting industry information in this area from Queensland underground mines. To date none have been designed taking into account underground measured levels of mine atmosphere DPM levels.

Levels of gaseous pollutants allowed in mine workplaces are well understood and measured underground by fixed electronic monitors, tube bundle measurements or hand held multi-gas monitors. Approaches to understanding what are acceptable levels of DPM pollutants in mine workplaces in Australia and overseas are not well understood and at a formative stage.

A Tag Board design exercise has been undertaken to examine implications of this approach of using directly measured mine atmosphere exhaust gas and DPM readings. The underground monitoring used in the Tag Board design exercise was based on evaluation of DPM from various vehicles under working conditions. Tag Board Design needs to consider a number of issues.

- Who is being analysed? Is it the driver and personnel on moving vehicles travelling in and out of the panel? Or is it the crew within the panel and particularly those at the face?
- What is the relationship between “make” of DPM from a particular vehicle and airflow for dilution within the travelling airway?

The DPM breathed by vehicle occupants will depend on the vehicle engine’s exhaust output, the airflow ventilation route, the roadway and whether it is uphill or downhill, whether the air is travelling with or against the vehicle direction, the air velocity as a function of the air quantity and vehicle’s travel speeds. Exhaust pollution effects can be significantly reduced if vehicles do not travel in convoy or close together. Effects can be reduced if vehicles do not travel at the air velocity and either travel slower than ventilation air velocity so that the plume of exhaust travels faster than the vehicle or alternatively travel faster than ventilation air velocity so that the plume of exhaust is left behind.

The effect of DPM on crew members at a working face is important. All DPM contaminant exhausted while a vehicle is in a section passes through the working place except for leakage that short circuits through stoppings and other ventilation control devices. Crew members are thus affected by a vehicle’s DPM “make” which is best determined by testing it during normal working conditions. This should take into operational conditions such as road conditions, road gradient up or down, engine revving or idling periods and so on. From this a particular vehicle’s DPM operational signature can be determined.

The relationship between “make” of DPM from a particular vehicle and airflow for dilution within the travelling airway can be determined as follows.

- A vehicles DPM pollution in the mine airway is measured in mg/m³ in a particular airway
- Air way ventilation quantity at that point is measured in m³/s
- DPM “make” is the product of the two i.e. mg/m³ x m³/s = mg/s

The effect of a vehicle’s make depends on air quantity in the ventilation split. Greater air quantity increases dilution. Tag Board design in considering the face crew members must have information on the following

- Average make of each vehicle that may be in the ventilation split (mg/s)
- The quantity of air available for dilution (m³/s)
- Maximum number of vehicles at a particular time (and which vehicles)
- The DPM pollutant level that is considered (by design, guidelines or regulations) to be the maximum (mg/m³) that is considered acceptable.

Tests were undertaken at Mine 4 over one day to assist in Tag Board design. The exercise produced DPM make values from underground measured values supported by mine workshop/ industry published data as shown in Table 3.
Table 3 - DPM Make of some test mine vehicle incorporating workshop and underground monitored values

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>Engine kW</th>
<th>Av Make</th>
<th>U/G Test 1 mg/s</th>
<th>U/G Test 2 mg/s</th>
<th>U/G Test 3 mg/s</th>
<th>U/G Test 4 mg/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toyota</td>
<td>55</td>
<td>3.05/9.21</td>
<td>0.08, idle</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SMV Drifty</td>
<td>63</td>
<td>2.97/5.94</td>
<td>2.14, idle</td>
<td>1.34, idle</td>
<td>2.59</td>
<td>-</td>
</tr>
<tr>
<td>Eimco, CAT 3306</td>
<td>Av 105</td>
<td>3.14/9.81</td>
<td>1.5, idle</td>
<td>9.02, rev</td>
<td>3.27</td>
<td>2.07</td>
</tr>
</tbody>
</table>

*Average and maximum make from SIMTARS workshop test industry database

Monitored values indicated
- The one Toyota reading was very low compared with workshop value. Further investigation from this one value is needed.
- Single Drifty outputs 2.0 to 3.0 mg/s in normal use. Good underground and workshop test agreement.
- Single Eimco also outputs 2.0 to 3.0 mg/s in normal use; more under heavy load. Good underground and workshop test agreement.

It was also found that convoy tests for two and three vehicles gave outputs that cumulatively agreed with figures for single vehicles.

Some conclusions for Tag Board Design for DPM requirement indicated that future tests should undertake more extensive tests with single and vehicles convoys and undertake more tests at extremes of operation eg, heavy loads, steep gradients and prolonged idling. Tests over longer routes on a more representative set of road surfaces; particularly more roads in “bad” condition should be undertaken. Some tests should be undertaken in a quieter period such as during night shift to reduce or eliminate interference from other (non test) vehicles and there should be some underground tests while vehicles are parked and idling. Testing for DPM requirements for Tag Board design have been undertaken over one day. The D-PDM real time monitors in mine static and moving positions gave good and consistent monitored results representative of the underground environment. Underground readings in general agree well with workshop tests. Recommendations were formulated on some additional tests to increase confidence in results.

The real time DPM monitor is being developed on the base of the successful PDM unit. The only other unit available in Australia for measuring directly mine atmosphere DPM is the NIOSH developed SKC impactor system. The SKC system delivers shift average results and not real time results. The SKC system results are analysed by the NIOSH 5040 method and the only Australian site for this analysis is the Singleton, New South Wales Coal Services Laboratory. During this research parallel underground SKC samples have been taken for comparison with the mine real time DPM monitor results. Under the SKC system the sample is drawn first through a respirable cyclone sampler and then through an impactor before passing onto a quartz filter. It can then be analysed for carbon; both the OC associated with the absorbed organic substances and EC from the soot cores themselves. TC is the sum of the OC and EC. TC according to Volkwein (2006) makes up consistently over 80 percent of the submicron DPM material that passes through the impactor in the SKC system. From various research and studies conducted so far, TC has been measured at over 80 percent of submicron DPM sample mass. Dabill (2005) states that comprehensive research has shown that over 95 percent of diesel particulate has an aerodynamic diameter of less than 1 μm, whereas virtually all coal dust has particles larger than 1 μm. Consequently by collecting the submicron fraction the coal dust is effectively eliminated.

Figure 11 shows results from the first three mine test series compared with SKC impactor collection determinations of EC and TC particulate shift average results taken in the particular mine at the same time. Close correlations were found for all cases and in particular for Mines 2 and 3. The results demonstrate that calibration relationships vary mine to mine due to differences in aspects such as mine atmospheric contamination, fuel type, engine maintenance and engine behaviour.

CONCLUSIONS

Two project areas of new real time monitoring development supported by ACARP grants in recent years have been discussed. The projects received substantial NIOSH support and are stories in practical application that have received considerable additional industry financial support, mine site testing and evaluation assistance. The paper has discussed how the monitors have performed within the underground mine environment in evaluating respirable dust and diesel particulate matter during the various phases of a production cycle. They have closely examined the influence of aspects of the mine ventilation system. Results have been compared to alternative industry pollutant measuring approaches. These monitors give the potential to improve understanding of the mine environment and to empower and educate operators in the control of their environment. Both monitoring approaches have application to coal and metalliferous surface mining operations in addition to the underground evaluations discussed.
ACKNOWLEDGEMENTS

The author acknowledges the support of ACARP and NIOSH in supporting initial projects that form the basis of this paper. They extend thanks to the various mine site managers, engineers and ventilation officers who supported the projects and the evaluation efforts undertaken across a diversity of mine conditions. Their efforts ensured that the principal development and mine site testing aims of the projects were accomplished and a significant contribution made to future mine health and safety in Australia.

REFERENCES


Volkwein, J.C.,219


MOISTURE CONTENT IMPACT ON THE SELF-HEATING RATE OF A HIGHLY REACTIVE SUB-BITUMINOUS COAL

B Basil Beamish and Timothy J Schultz

ABSTRACT: A New Zealand power station has been importing subbituminous coal, which has occasionally created a spontaneous combustion problem at the port storage facility. Samples of the coal have been tested using an adiabatic oven to determine the self-heating rate of the coal at various moisture contents ranging from as-received to dry. Initial self-heating rates from room temperature were higher for samples containing up to 75% of the as-received moisture content compared to dry coal. However, the overall time to reach thermal runaway increased with moisture content. This paper clearly shows the highly reactive nature of the subbituminous coal as all tests would have proceeded to ignition, even those performed at close to the as-received moisture content of the coal.

INTRODUCTION

The self-heating of coal is due to a number of complex exothermic reactions. Coal will continue to self-heat provided there is a continuous supply of oxygen and the heat generated is not dissipated. Moisture content can affect the self-heating rate of coal in two ways: changing the overall heat balance; and the rate of the oxidation reaction. There have been a number of investigations to help provide a better understanding these processes (Chamberlain, 1974; Humphreys and Richmond, 1987; Smith and Lazarra, 1987; Walters, 1996; Clemens and Matheson, 1996; Vance, Chen and Scott, 1996; Bhat and Agarwal, 1996).

Many studies have also been completed examining how differing levels of coal moisture content affect the self-heating rate of coals. These have produced conflicting results. Sondreal and Ellman (1974) found that for a lignite sample there was a critical moisture content at which the rate of oxidation reached a maximum. This finding is disputed by Bhat and Agarwal (1996) who claimed that the test process used changed the subsequent low-temperature oxidation behaviour of the coal. Clemens and Matheson (1996) also reported that samples of low-rank coals containing varying amounts of moisture experienced different rates of initial self-heating, with some moist samples oxidising at a faster rate than the dry samples. Beamish and Hamilton (2005) found that for a subbituminous coal from the Callide Basin, self-heating was inhibited until the coal had lost approximately half its moisture holding capacity.

Adiabatic testing procedures have been used at The University of Queensland to study a highly reactive subbituminous coal from Indonesia. This paper presents results from the adiabatic self-heating rate tests that show the effect moisture has on the coals reactivity.

EXPERIMENTAL PROCEDURE

Coal samples

The coal sample used for test work was taken from a stockpile of imported Indonesian coal at the New Zealand port of Tauranga. The sample was sent to The University of Queensland's Spontaneous Combustion Testing Laboratory, with each of the coal lumps individually wrapped in cling wrap, and stored in an air-tight sealed bucket at room temperature until the commencement of testing.

Preparation and testing of samples

The coal was split into six 250 g samples of -30 mm coal and placed into frozen storage until they were required for testing. Prior to testing, each 250 g sample was ground and screened to produce 150 g of -212 μm coal and flushed under nitrogen.

The first test, INDO1A, was prepared and tested under normal R70 conditions (dry basis), which required the test sample to be dried at 110°C for 16 hours. After this it was transferred to the adiabatic oven and the test was commenced from a start temperature of 40°C. A full description of the adiabatic testing procedure is outlined by Beamish, Barakat and St George (2000).

The remaining tests were conducted at varying moisture contents to investigate the effects of moisture content on the self-heating rate from a start temperature of ~25°C. To do this, the drying times in the laboratory oven were varied as required to allow the coal sample to attain the desired moisture content. This included a test on dry coal for comparison with the original R70 test.

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RESULTS AND DISCUSSION

Coal quality data are contained in Table 2. As the coal was of low rank, the gross calorific value on a moist, mineral matter free basis (mmmf) was used to determine the ASTM rank classification of the coal (ASTM, 2004). The coal is borderline sub-bituminous B/C.

Table 2 - Coal quality and proximate data of INDO coal sample

<table>
<thead>
<tr>
<th></th>
<th>INDO1A</th>
<th>INDO2A - E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture (%) ad</td>
<td>17.7</td>
<td>17.7</td>
</tr>
<tr>
<td>Moisture (%) ar</td>
<td>25.2</td>
<td>24.3</td>
</tr>
<tr>
<td>Ash Content (%) db</td>
<td>2.0</td>
<td>1.8</td>
</tr>
<tr>
<td>Volatile Matter (%) dmmf</td>
<td>51.6</td>
<td>51.6</td>
</tr>
<tr>
<td>Calorific Value (Btu/lb, mmfmf)</td>
<td>9755</td>
<td>9755</td>
</tr>
</tbody>
</table>

The $R_{70}$ self-heating rate index of the coal is determined by finding the average self-heating rate between 40°C and 70°C, in units of °C/h as shown in Figure 1 This was found to be 28.57°C/h, which compares favourably with an earlier result (35.11°C/h) for a different sample of the coal.

Moisture contents for the INDO2A-E replicates are contained in Table 2. These are approximates for: fully dried coal; 75% as-received moisture; 50% as-received moisture; 25% as-received moisture; and approximately as-received moisture.

Initial self-heating rate curves for the INDO2A-E replicates are shown in Figure 2 A value for the initial self-heating rate (from the room temperature start through to 70°C), has been calculated for each of the coal moisture states (Table 3) The INDO2D test sample, containing 7.3% moisture (approximately 25% of as-received moisture), has the highest value of all the tests. This suggests that there is an optimum moisture content for the coal where the initial rate of self-heating is considerably enhanced (Figure 3) It is also evident from Figure 3 that even the sample with 75% of the as-received moisture content present has an initial self-heating rate faster than the dried coal. However, the sample with close to the as-received moisture content had a slower initial self-heating rate than the dry coal.

Table 3 - Moisture content data for INDO2A-E replicates

<table>
<thead>
<tr>
<th>Test Sample</th>
<th>As Received Moisture (%)</th>
<th>Test Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>INDO2A</td>
<td>24.3</td>
<td>0.0</td>
</tr>
<tr>
<td>INDO2D</td>
<td>24.3</td>
<td>7.3</td>
</tr>
<tr>
<td>INDO2C</td>
<td>24.3</td>
<td>12.5</td>
</tr>
<tr>
<td>INDO2B</td>
<td>24.3</td>
<td>16.7</td>
</tr>
<tr>
<td>INDO2E</td>
<td>24.3</td>
<td>24.0</td>
</tr>
</tbody>
</table>

The relationship between initial self-heating rate and moisture content is very similar to that found by Vance, Chen and Scott (1996) for a sub bituminous B coal from New Zealand. Work by Clemens and Matheson (1996) on New Zealand coals of similar rank produced similar findings and they attributed the increased self-heating of the moist coal to the presence of tightly bound moisture generating radical sites in the coal (where oxidation occurs) that are more reactive than those derived from the fully dried coal.
Table 4 - Initial self-heating rates for INDO2A-E replicates

<table>
<thead>
<tr>
<th>Test Sample</th>
<th>Test Moisture Content (%)</th>
<th>Self-Heating Rate (°C/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>INDO2A</td>
<td>0.0</td>
<td>12.92</td>
</tr>
<tr>
<td>INDO2D</td>
<td>7.3</td>
<td>21.62</td>
</tr>
<tr>
<td>INDO2C</td>
<td>12.5</td>
<td>17.07</td>
</tr>
<tr>
<td>INDO2B</td>
<td>16.7</td>
<td>16.07</td>
</tr>
<tr>
<td>INDO2E</td>
<td>24.0</td>
<td>10.87</td>
</tr>
</tbody>
</table>

Figure 1 - $R_{70}$ self-heating rate index for INDO1A sample

Figure 2 - Initial self-heating rate curves for the INDO2A-E replicates at different moisture content
In contrast, Beamish and Hamilton (2005) found a different relationship between initial self-heating rate and moisture content for a sub-bituminous A coal from Boundary Hill tested from a 40°C start temperature. The results of this work are compared with the Indonesian coal in Figure 3. There is a substantial difference between the two coals. The moisture activation observed in lignites and the lower rank sub-bituminous coals (B and C) may not apply to higher rank coals from sub-bituminous A upwards. Alternatively, the Boundary Hill coal has a high inertinite content that is not present in the Indonesian coal, which is rich in vitrinite. This may also explain the discrepancies noted in the literature between various moisture studies on coal self-heating.

Despite the variations in the initial self-heating rates exhibited by the changes in moisture content within the INDO2A-E replicates, it was found that the time to thermal runaway (at ~160°C) increased with moisture content (Table 4) with the fully dry INDO2A sample achieving thermal runaway in just under four hours, whilst the INDO2E sample with close to as-received moisture taking just over 23 hours. The self-heating curves presented in 4 shows the extent to which variations in initial moisture content can affect the overall self-heating behaviour of the coal sample, when starting from room temperature (~25°C). The increasing time to thermal runaway was attributed to the amount of time taken for the coal to boil off any residual moisture, thus creating a heat loss and affecting the overall heat balance of the self-heating process. Consequently, the drier the coal the faster it will reach ignition.

**TABLE 5 - TIME TO THERMAL RUNAWAY FOR INDO2A-E REPLI CATES**

<table>
<thead>
<tr>
<th>Test Sample</th>
<th>Test Moisture Content (%)</th>
<th>Time to Thermal Runaway (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>INDO2A</td>
<td>0.0</td>
<td>3.86</td>
</tr>
<tr>
<td>INDO2D</td>
<td>7.3</td>
<td>5.22</td>
</tr>
<tr>
<td>INDO2C</td>
<td>12.5</td>
<td>8.65</td>
</tr>
<tr>
<td>INDO2B</td>
<td>16.7</td>
<td>12.15</td>
</tr>
<tr>
<td>INDO2E</td>
<td>24.0</td>
<td>23.20</td>
</tr>
</tbody>
</table>

![Figure 3 - Comparison between initial self-heating rates and moisture contents of INDO2 replicates and Boundary Hill coal](image-url)
CONCLUSIONS

Adiabatic testing of a highly reactive sub-bituminous coal from Indonesia has resulted in an $R_{70}$ self-heating rate index value of 28.57°C/h. Additional testing of this coal at different moisture contents from an approximate 25°C starting temperature has produced self-heating results that are similar to previous work on coals of this rank.

Samples containing up to approximately 75% of the "as-received" moisture content exhibited faster initial self-heating rates than the fully-dried sample. This was attributed to the enhanced reactive site theory proposed by Clemens and Matheson (1996). A maximum initial self-heating rate occurred at approximately 25% of the "as-received" moisture content. These results are in contrast to a similar study on coal from the Callide Basin, which produced initial self-heating rates of zero until the coal moisture content fell to approximately half the moisture holding capacity.

The overall time to thermal runway (at ~160°C) however, increased with moisture content, which was attributed to the extra time required for boiling off any residual moisture contained within the sample.

These findings suggest that increasing the moisture content of this coal would not eliminate the risk of the coal self-heating whilst being transported and stored, but the self-heating data does indicate that keeping the moisture content of the coal high will increase the time to ignition. This would have to be considered on a cost benefit basis, and clearly the simplest solution is to use the coal as soon as possible.

ACKNOWLEDGEMENTS

The authors would like to thank Keith Hopkins at Genesis Energy for supplying and shipping the coal sample from New Zealand for testing.
REFERENCES


DEVELOPMENT OF A SITE SPECIFIC SELF-HEATING RATE PREDICTION EQUATION FOR A HIGH VOLATILE BITUMINOUS COAL

B Basil Beamish and Wade Sainsbury

ABSTRACT: The R70 test was performed on a series of coal samples taken from different locations in a US longwall mine. The values obtained produced a definite relationship between R70 and ash content, with the exception of one anomalously low result. An ash analysis of the sample showed that it had a high sodium (Na2O) content in response to the presence of the sodium zeolite mineral, analcime. A multiple regression of the R70, ash content and sodium (Na2O) content of the samples produced a self-heating rate prediction equation with an R^2 of 0.98. This equation can now be used to predict the R70 self-heating rate of the coal at any location throughout the mine, thus assisting with hazard management planning.

INTRODUCTION

Low-temperature oxidation of coal results in an exothermic reaction and without sufficient dispersion of the heat generated will eventually result in spontaneous combustion. This unwanted outcome can cause huge losses in revenue and more importantly, cause major problems with safety. In underground mining operations the combination of in-seam gas, inappropriate ventilation networks and self-heating areas of coal can equate to a catastrophic disaster. Examples of these events in Australia are recorded by Ham (2005).

The parameters that control a coal’s propensity for self-heating have been the subject of many investigations. Relationships between coal properties and self-heating indices have been published in a number of studies (Humphreys, Rowlands and Cudmore, 1981; Moxon and Richardson, 1985; Singh and Demirbilek, 1987; Barve and Mahadevan, 1994; Beamish, Barakat and St. George, 2000, 2001). Humphreys, Rowlands and Cudmore (1981) found a simple relationship between the coal self-heating index parameter, R70 and coal rank. However, research by Beamish, Barakat and St. George (2001) and Beamish (2005) on New Zealand and Australian coals covering a wider range of coal ranks showed that the R70 coal self-heating rate relationship with rank is non-linear. Beamish and Blazak (2005) also showed that R70 values decrease significantly with increasing mineral matter content, as defined by the ash content of the coal.

This paper presents the development of a site specific equation used for the prediction of R70 self-heating rate of a high volatile bituminous coal that is being mined by longwall methods in the United States (US). The site specific nature of coal self-heating has been identified by earlier work on Australian coals using the R70 test procedure (Beamish et al., 2005; Beamish and Clarkson, 2006). However, this is the first time that the R70 test procedure has been applied to US coals.

EXPERIMENTAL PROCEDURE

Coal samples

The coal samples used for test work were collected from the workings of an operating longwall coal mine in the United States. These samples were sent to The University of Queensland’s Spontaneous Combustion Testing Laboratory, with each of the coal lumps individually wrapped in cling wrap, and stored in an air-tight sealed bag that was placed in the laboratory freezer until the commencement of testing.

R70 Test Procedure

The full adiabatic oven testing procedure is outlined in (Beamish, Barakat and St George, 2000). In preparation for the test, the coal samples are taken out of the freezer and allowed to thaw for one hour. Once the samples are thawed they are crushed and sieved to achieve a particle size of < 212 µm. This process is to be completed in as little time as possible to reduce oxidation on the fresh surface of the coal. The R70 test is carried out on a dry basis therefore all the samples are dried. 150 g of the sample is placed in a 750mL flask then in the drying oven and all the oxygen removed. The sample is kept in the oven with a constant nitrogen flow of 250 mL/min. Then it is heated to 110°C and left to dry for 16 hours. After the drying process is complete, the sample is transferred to the 450 mL reaction vessel and placed in the adiabatic oven. Here it is allowed to stabilise at 40°C in a nitrogen-rich atmosphere. Once this was achieved,
oxygen is then supplied to the coal at a rate of 50mL/min. For the duration of the test, the oven is put on remote monitoring where the change in temperature of the coal is followed by the oven to minimise any heat losses from the system and the data is recorded by a computer logging system. Once the coal temperature reaches 160°C, the supply of oxygen is cut off and the oven heating elements are disengaged. The sample is then allowed to cool before being removed and the equipment cleaned and prepared for the next test.

RESULTS AND DISCUSSION

R70 Results and Ash Content Relationship

Adiabatic self-heating curves for each of the seven samples tested are shown in Figure 1. The R70 value is determined as the average self-heating rate from the starting temperature (~40°C) till the coal reaches 70°C and is expressed in units of °C/h. A plot of the R70 values against their respective ash contents is shown in Figure 2. There is a strong linear relationship evident for six of the seven samples with an R2 value of 0.97. As described by Beamish and Blazak (2005), generally as the ash content increases for a given rank of coal the R70 value decreases. In earlier work this has been attributed to the mineral matter in the coal acting as a heat sink (Humphreys, Rowlands and Cudmore, 1981; Smith, Miron and Lazzara, 1988) and thus lowering the self-heating rate of the coal. However, more recent work by Beamish and Arisoy (2008) points to the possibility of other physico-chemical mechanisms causing the decrease in self-heating rate of the coal. The slope of the relationship shown in Figure 2 is much steeper than those presented by Beamish et al. (2005) and Beamish and Clarkson (2006) for Australian high volatile bituminous coals. This steep slope is not consistent with a simple heat sink effect.

Sample 6A appears to be an outlier compared to the rest of the samples tested. This cannot be attributed to repeatability error, as samples 3A and 3B have the same ash content (6.5%) and produced R70 values of 3.09 and 3.01°C/h respectively. These values are well within the normal repeatability limits for testing of ±5%.

One possible explanation for the dramatic decrease in self-heating rate for this sample could be a difference in coal type due to a change in maceral composition. However, a Suggate rank plot (Suggate, 1998 and 2000) of the samples showed no variation in coal type between them. The only possibility left is that there is a different mineral composition present in this sample that is causing the decrease in R70.

Ash Analysis

In an effort to explain the variance observed in the R70 result obtained for sample 6A an ash analysis was conducted to identify possible mineralogical associations. The ash analysis uses a wavelength dispersive x-ray fluorescence spectrometric method which can determine concentrations of silicon, aluminium, iron, calcium, magnesium, sodium, potassium, titanium, manganese, phosphorous, sulphur, strontium, barium, zinc and vanadium.

A duplicate of each sample of coal was sent to a registered laboratory for ash analysis according to the relevant Australian Standards (AS1038.3 2000 and AS1038.14.3 1999). The results from the seven different samples have been collated in Table 1. Each sample was tested for the presence of 15 standard elements and is displayed as a percentage dry basis in oxide form.

The most striking feature about the ash analysis results for sample 6A (Table 1) is the very high sodium content (10%). This is at least three times the levels of the other samples. Having obtained this possible link to the cause of the low R70 value for the sample, the next step was to identify the mineral responsible for the sodium. A sample of the coal was analysed using X-Ray Diffraction (XRD) and the results obtained showed the presence of the sodium zeolite mineral, analcime (NaAlSi2O6·H2O). Initial scanning electron microscopy analysis has confirmed this and low temperature ashing of the sample is still in progress to establish any other mineral assemblage associations. The results of this additional work will be published at a later date, including a possible genesis for the presence of the analcime.
Figure 1 - Adiabatic self-heating curves for a high volatile bituminous coal

\[ y = -0.6162x + 7.027 \]
\[ R^2 = 0.9696 \]

Figure 2 - Relationship between R_{70} self-heating rate and ash content

\[ y = -0.6162x + 7.027 \]
\[ R^2 = 0.9696 \]
Table 1 - Ash Analysis Results

<table>
<thead>
<tr>
<th>Ash Composition (% db)</th>
<th>3A</th>
<th>3B</th>
<th>4</th>
<th>5A</th>
<th>5B</th>
<th>6A</th>
<th>6B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon (SiO₂)</td>
<td>56.0</td>
<td>56.7</td>
<td>70.0</td>
<td>49.0</td>
<td>53.0</td>
<td>50.5</td>
<td>54.5</td>
</tr>
<tr>
<td>Aluminium (Al₂O₃)</td>
<td>30.2</td>
<td>31.0</td>
<td>18.1</td>
<td>28.0</td>
<td>27.0</td>
<td>25.5</td>
<td>23.5</td>
</tr>
<tr>
<td>Iron (Fe₂O₃)</td>
<td>4.6</td>
<td>4.8</td>
<td>4.7</td>
<td>8.3</td>
<td>5.4</td>
<td>5.2</td>
<td>4.0</td>
</tr>
<tr>
<td>Calcium (CaO)</td>
<td>0.8</td>
<td>0.82</td>
<td>1.2</td>
<td>4.5</td>
<td>4.1</td>
<td>4.0</td>
<td>4.9</td>
</tr>
<tr>
<td>Magnesium (MgO)</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.9</td>
<td>1.5</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Sodium (Na₂O)</td>
<td>2.7</td>
<td>3.0</td>
<td>1.6</td>
<td>3.4</td>
<td>2.5</td>
<td>10.0</td>
<td>3.5</td>
</tr>
<tr>
<td>Potassium (K₂O)</td>
<td>0.47</td>
<td>0.48</td>
<td>0.41</td>
<td>0.47</td>
<td>0.63</td>
<td>0.44</td>
<td>0.38</td>
</tr>
<tr>
<td>Titanium (TiO₂)</td>
<td>1.5</td>
<td>1.7</td>
<td>1.7</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>Manganese (Mn₃O₄)</td>
<td>0.02</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Phosphorous (P₂O₅)</td>
<td>0.14</td>
<td>0.15</td>
<td>0.12</td>
<td>0.56</td>
<td>1.50</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>Sulphur (SO₃)</td>
<td>0.61</td>
<td>0.43</td>
<td>0.8</td>
<td>2.6</td>
<td>2.7</td>
<td>1.9</td>
<td>3.5</td>
</tr>
<tr>
<td>Strontium (SrO)</td>
<td>0.50</td>
<td>0.35</td>
<td>0.30</td>
<td>0.40</td>
<td>0.19</td>
<td>0.51</td>
<td>0.01</td>
</tr>
<tr>
<td>Barium (BaO)</td>
<td>0.35</td>
<td>0.35</td>
<td>0.30</td>
<td>0.30</td>
<td>0.34</td>
<td>0.45</td>
<td>0.23</td>
</tr>
<tr>
<td>Zinc (ZnO)</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Manganese (V₂O₅)</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Multiple regression analysis

There appears to be two dominant factors affecting the $R_{70}$ self-heating rate values of these samples, ash content and the amount of sodium present in the ash. To determine the influence these parameters are having a multiple regression analysis was performed with the $R_{70}$ value as the dependent factor. The regression equation obtained is:

$$R_{70} \, (^\circ \text{C}/\text{h}) = -0.6351 \times \text{Ash} - 0.1767 \times \text{Na}_2\text{O} + 7.63$$

($R^2 = 0.98$)

where, both Ash and Na₂O are in dry weight percent.

A comparison of the actual and predicted self-heating rates found that only a minor variance existed between the two values. The confidence of this equation is currently average, however a larger dataset would allow a much more precise equation to be created. Table 2 contains the calculated residual values for each of the samples. It can be seen that the difference ranges from 0.003 to 0.095 °C/h, which is extremely low and well within the repeatability limits of the test. Therefore, this equation can be used throughout the mine with an acceptable degree of confidence for predicting the $R_{70}$ self-heating rate of the coal.

Table 2 - Comparison Actual and Predicted $R_{70}$

<table>
<thead>
<tr>
<th>Sample</th>
<th>Actual $R_{70}$ ($^\circ \text{C}/\text{h}$)</th>
<th>Predicted $R_{70}$ ($^\circ \text{C}/\text{h}$)</th>
<th>Residual ($^\circ \text{C}/\text{h}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>3.09</td>
<td>3.02</td>
<td>0.069</td>
</tr>
<tr>
<td>3B</td>
<td>3.01</td>
<td>2.96</td>
<td>0.042</td>
</tr>
<tr>
<td>4</td>
<td>3.39</td>
<td>3.46</td>
<td>0.079</td>
</tr>
<tr>
<td>5A</td>
<td>4.32</td>
<td>4.23</td>
<td>0.089</td>
</tr>
<tr>
<td>5B</td>
<td>3.66</td>
<td>3.75</td>
<td>0.095</td>
</tr>
<tr>
<td>6A</td>
<td>3.04</td>
<td>3.06</td>
<td>0.025</td>
</tr>
<tr>
<td>6B</td>
<td>2.94</td>
<td>2.94</td>
<td>0.003</td>
</tr>
</tbody>
</table>
CONCLUSIONS

Adiabatic testing has been performed on a series of high volatile bituminous coal samples taken from different locations through an underground longwall mining operation. The values obtained produced a definite linear relationship between R70 and ash content. A simple heat sink effect mechanism is not evident as the decreasing self-heating rate trend for the samples is much too steep for this.

An ash analysis of the coal provided a quantitative breakdown of the major inorganic constituents within the samples. These results showed a strong negative association with sodium (Na₂O) in one of the samples which did not conform to the observed simple linear R70 and ash content relationship. The sodium in the sample is linked to the presence of the mineral analcime, which may be acting as a natural inhibitor to the coal oxidation process and thus reducing the self-heating rate.

A multiple regression of the R70, ash content and sodium (Na₂O) content produced a self-heating rate prediction equation with an R² of 0.98. Similar site specific relationships are being developed for other mines throughout Australia and overseas, which can be used to assist with the mine’s hazard management planning.

ACKNOWLEDGEMENTS

The authors would like to thank mine site personnel for providing an excellent range of samples to be tested. The mine name has been withheld for confidentiality reasons. UniQuest Ltd and Simtars are also thanked for granting permission to publish these results.

REFERENCES


A STUDY OF THE FORMATION OF HYDROGEN PRODUCED DURING THE OXIDATION OF BULK COAL UNDER LABORATORY CONDITIONS

William K Hitchcock1, B Basil Beamish1 and David Cliff2

ABSTRACT: A number of studies of the oxidation of coal using The University of Queensland’s two-metre, 62L test rig have been carried out over the past few years. The rig simulates a semi-adiabatic environment radially and allows gas samples to be taken along its length and from the exhaust stream. This enables the generation of a gas and temperature profile across a coal self-heating zone. As the state of spontaneous combustion in underground coal mines is usually inferred from gas samples taken remote to the heatings these laboratory studies offer important insights into the mechanisms of gas formation during coal self-heating events. In particular much emphasis is placed upon the presence of and concentration of any hydrogen. This paper reports the preliminary findings from a test where such gas samples were taken. The bulk of the hydrogen appears to be generated downstream from the hot spot where the coal is at approximately 100°C and there is no free oxygen.

INTRODUCTION

Coal self-heating leading to spontaneous combustion continues to pose a significant hazard during the mining of coal. A recent example of this is Southland Colliery in December 2003, where a heating progressed to open fire forcing the mine to be closed. Another example is the spontaneous combustion event at Newstan Colliery 2005-06, that spanned over twelve months and cost many millions of dollars to control. Unfortunately, the heterogeneous nature of coal and the contributing factors that control whether heat is gained or lost from the coal/oxygen system make it difficult to predict the onset of a heating with any confidence.

As part of the management strategy for spontaneous combustion at all Australian underground coal mines, there is a requirement to have in place trigger action response plans (TARPS) which rely heavily on gas monitoring and analysis of the mine atmosphere. These plans make use of gas indicators such as CO make, Graham’s ratio, hydrogen production etc (Cliff, Rowlands and Sleeman, 1996), which act as guides to the stage that a coal self-heating may have reached. In particular, significant amounts of hydrogen are regarded as indicating an advanced heating.

The use of these indicators has been developed from research on evolved gas studies, in particular the work by Pursall and Ghosh (1965) and Chamberlain, Hall and Thirlaway (1970). More recent studies have been conducted by Street, Smalley and Cunningham (1975), Hurst and Jones (1985) and Wang, Dlugogorski and Kennedy (2002). All of these studies have used test methods involving grams of pulverised coal and air flow rates in the order of mL/min, resulting in high airflow to mass ratio conditions. However, these are not the conditions that are encountered in the mine environment.

Bulk coal self-heating tests have been limited due to the expense and time taken to obtain results. Some success has been obtained with various column-testing arrangements (Li and Skinner, 1986; Stott and Chen, 1992; Akgun and Arisoy, 1994; Arief, 1997), but the equipment used has not gained wide acceptance. A laboratory has been established within the School of Engineering at The University of Queensland (UQ) that uses a two-metre column to conduct a practical test capable of providing reliable gas evolution and temperature data on coal self-heating. The column allows not only the gas evolution at the hot spot location to be examined, as small-scale tests do, but also allows the examination of gas evolution downstream from the hot spot. This paper presents some of the gas results from a test on a high volatile A bituminous coal from the Bowen Basin using the two-metre column.

COLUMN SELF-HEATING

Equipment

Beamish et al., (2002) describe the basic operation of the UQ two-metre column, which has a 62 L capacity, equating to 40 – 70 kg of coal depending upon the packing density used. The coal self-heating is monitored using eight evenly spaced thermocouples along the length of the column that are inserted into the centre of the column. A port for gas extraction is located adjacent to each thermocouple. Eight independent heaters correspond to each
of these thermocouples and are set to switch on and off according to balancing equations which ensure that heat losses are minimised and semi-adiabatic conditions are maintained radially.

Figure 1 shows a schematic of the UQ column.

![Figure 1 - Schematic of UQ two-metre column](image)

**Sample Preparation**

A coal sample was obtained from a Bowen Basin coal mine for testing in the UQ two-metre column. The coal was crushed to an average particle size of less than 12.7 mm. This facilitated easy handling of the sample, particularly with regards to loading the column and insertion of the coal thermocouples. Three subsamples were taken at this stage to obtain data on the as-received moisture content of the coal, which was determined to be 6.7%. Samples were also taken at this stage to establish the $R_{70}$ self-heating rate of the coal.

**Test Procedure**

The coal was loaded into the column with three 20 L plastic buckets. A total of 56 kg of coal was loaded. The lid was then secured and nitrogen flushed through the column at 0.5 L/min and the heaters used to set the starting coal temperature, which in this case was initially 40°C. Once the coal temperature had stabilised the nitrogen was switched off and air was then introduced to the coal at a flow rate of 0.5 L/min. A computer recorded all the data at ten-minute increments. The column has several safety devices including computer-controlled trips on the external heaters. These were set to ensure maximum safety during operation of the column.

As the column test progressed gasbag samples were collected from the exhaust and ports located along the length of the column. A peristaltic pump was used to suck samples from the ports. This pump had a low flow rate relative to the column so that sampling does not disturb the normal gas flow within the reactor and is designed such that there is no gas leakage. The gas samples were later analysed by Simtars using standard gas chromatography. In total there were four gas profiles completed throughout the test.

**RESULTS OF $R_{70}$ AND COLUMN TESTING**

**$R_{70}$ value of the column sample**

The $R_{70}$ testing procedure is described by Beamish, Barakat and St George (2001). Essentially, a 150 g coal sample is crushed to less than 212 μm, dried under nitrogen at 110°C and then tested under oxygen in an adiabatic oven. The $R_{70}$ value is a simply the average rate of heating of the coal between 70°C from a starting temperature of 40°C and is expressed in units of °C/h. Figure shows the self-heating curve obtained in the UQ adiabatic oven for the sample taken whilst loading the column. The $R_{70}$ value determined from this test was 0.52°C/h. This places the coal on the borderline between the ‘low’ and ‘medium’ propensity to spontaneous combustion categories.
Figure 2 - Adiabatic self-heating curve for a Bowen Basin high volatile A bituminous coal

Column Testing

The hot spot initially developed at the downstream end of the column, before moving forwards towards the air source. This is typical of all column tests and is consistent with numerical modelling of spontaneous combustion. A total of four gas profiles were taken during the test. The temperature profiles of the column at the time of each gas profile are shown in Figure 3.

Gas evolution in response to coal oxidation and hot spot development

Table 1 details which locations were sampled for each column profile. These were determined based on the location and severity of the hot spot at the time of sampling. It should be noted that the gas sample from each port is the sum of all the gas evolution that has occurred prior to the air stream reaching that point.

For purposes of clarity, only the data for gas profiles 2 and 4 is presented. Figures 4 and 5 and Figures 7 and 8 respectively show the temperature and oxygen profiles. It can be seen that the hot spot strips most of the oxygen from the airstream and that on the downstream side of the hot spot the atmosphere is very oxygen depleted. This is consistent with what has been observed in small-scale test work which indicates that once the hot spot reaches the temperature region of 150°C - 200°C it will strip most of the oxygen from the atmosphere (Chamberlain, Hall and Thirlaway 1970; Street, Smalley and Cunningham 1975; Hollins 1995; Cliff, Bell and O’Beirne 1991).
Table 1 - Table of gas sample locations

<table>
<thead>
<tr>
<th></th>
<th>Exhaust</th>
<th>Port 9</th>
<th>Port 10</th>
<th>Port 11</th>
<th>Port 12</th>
<th>Port 13</th>
<th>Port 14</th>
<th>Port 15</th>
<th>Port 16</th>
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<tr>
<td>Profile 1</td>
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<td>-</td>
<td>-</td>
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<td>-</td>
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<td>yes</td>
<td>-</td>
</tr>
<tr>
<td>Profile 2</td>
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<td>-</td>
<td>-</td>
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<td>-</td>
<td>yes</td>
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<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>-</td>
</tr>
<tr>
<td>Profile 4</td>
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<td>-</td>
<td>yes</td>
<td>-</td>
<td>yes</td>
<td>-</td>
<td>yes</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 4 - Temperature profile 2 showing zones 1 and 2 based on hot spot location

Figure 5 – Gas profile 2 showing oxygen and hydrogen concentrations
Figure 6 – Gas profile 2 showing CO and CO₂ concentrations

Figure 7 - Temperature profile 4 showing zones 1 and 2 based on hot spot location

Figure 8 - Gas profile 4 showing oxygen and hydrogen concentrations
Figures 6 and 9 show the CO and CO₂ production for each profile. It is evident that these gases are produced by oxidation at the hot spot. Once again this is consistent with small-scale tests. Evidently, there are two distinct zones within the column. The first, Zone 1, is the region before and up to the hot spot which is undergoing oxidation reactions. This then transitions into Zone 2, which is located after the hot spot and is oxygen deficient. It should be noted that the oxygen depletion is not balanced by the production of oxides of carbon, i.e. there is a net oxygen absorption by the coal.

The hydrogen content in Figures 5 and 8 shows minimal amounts of hydrogen are produced in the Zone 1 region but there is significant hydrogen production throughout Zone 2. Small-scale tests on Bowen Basin coals conducted by Simtars show that hydrogen is only produced in significant amounts once the temperature range of 250°C - 325°C is reached whereupon the production rate ramps up significantly (Cliff, Bell and O’Beirne 1991). Small concentrations of hydrogen were detected at temperatures in excess of 100°C. The Simtars tests had an air flow to coal mass ratio ranging between 0.035 mL/min/g to 1 mL/min/g. Street, Smalley and Cunningham (1975) showed that, depending on rank and air flow to coal mass ratio, the temperature at which hydrogen was first produced (detected) could be below 100°C but could be as high as 250°C. For these studies the air flow to mass ratios ranged between 1.79 mL/min/g and 0.75 mL/min/g. It was observed that the lower the air flow to coal mass ratio the higher the appearance temperature of the hydrogen.

The work completed by Chamberlain, Hall and Thirlaway (1970) with an air flow to coal mass ratio of 1.6 mL/min/g showed hydrogen being initially produced at 70°C and then ramping up from 100°C onwards. This is consistent with Street, Smalley and Cunningham (1975). The column has an air flow to coal mass ratio of 0.009 mL/min/g which indicates that based on the small-scale research that hydrogen should not be detected in significant quantities until temperatures in excess of 300°C are reached. The results obtained from the two-metre column contradict this, generating the highest hydrogen concentrations of any laboratory test. Further these column results suggest that in a bulk coal situation, the majority of the hydrogen is in fact produced downstream of the hot spot where the coal is relatively cool i.e. around 100°C, not in the active oxidation zone. The temperature of the coal in this region suggests that the coal at this point is still evaporating moisture. This implies the majority of hydrogen production in a mining situation is not necessarily related to the temperature or intensity of the hot spot oxidation but is in fact more dependent on the amount of hot/warm moist coal located downstream from the hot spot.

Small-scale tests have shown that coal does not produce significant amounts of hydrogen at these temperatures under pyrolysis conditions. Therefore, there must be a catalyst involved in the production of the hydrogen. Work completed by Nehemia, Davidi and Cohen (1999) has shown that formaldehyde may be the precursor organic volatile that produces hydrogen with the coal acting as a catalyst. Fourier Transform Infrared (FTIR) analysis of coal has shown that aldehyde functional groups are part of the coal structure (Tognotti et al., 1991). Chamberlain, Barrass and Thirlaway (1976) showed that dry, crushed coal provided that sufficient oxygen was present would amongst other gases, produce acetaldehyde. Production reached a plateau at approximately 70°C, however, a second increase occurred above 130°C. This suggests that aldehyde groups may be precursors for hydrogen production and as such experiments should be conducted to examine this.
CONCLUSIONS

Significant quantities of hydrogen production from bulk-coal self-heating have been recorded. The majority of the hydrogen is not generated at the hot spot but in the oxygen depleted downstream region. Figures 5 and 8 show that the hydrogen production is not necessarily related to the temperature of the hot spot, but is related to how much coal is downstream from the hot spot which is at approximately 100°C. Considering significant increased hydrogen production in an underground atmosphere is regarded as indicating advanced oxidation this research has important implications for how mine atmospheres should be interpreted.

REFERENCES

IDENTIFICATION OF SPONTANEOUS COMBUSTION PRONE ZONES IN LONGWALL TOP COAL CAVING GOAFS

Jun Xie¹,², Sheng Xue² and Weimin Cheng¹

ABSTRACT: Longwall top coal caving (LTCC) mining method has been used to extract thick coal seams in China. Unfortunately the method has also brought with it an increased risk of spontaneous combustion (sponcom) in active LTCC goafs. It is therefore critical to identify the sponcom prone zones in LTCC goafs so that remedy measures can be taken to prevent sponcom from occurring. One of the successful methods to identify the sponcom prone zones is through the combination of field measurements of temperatures and oxygen concentrations inside a LTCC goaf and numerical modeling. A new technique has been developed specifically to enable the field measurements inside LTCC goafs to be easily undertaken. Presented in this paper are the description of such technique and its successful application in Xinglongzhuang coal mine of Yankuang Group, China.

INTRODUCTION

The LTCC system of mining thick coal seams is a productive and cost effective method, which has been widely used to extract thick coal seams in China. However the application of the method has also brought with it an increased risk of sponcom in active LTCC goafs because of the large caving zones formed and some fragmented coal left in the goaf. It is therefore critical to identify the sponcom prone zones in LTCC goafs so that measures can be taken to prevent sponcom from occurring.

Due to the inaccessible and complex nature of LTCC goafs, it is very difficult to make direct measurements inside the goaf although a number of attempts have been made with limited success (Luo, 1998; Xu, 2001; Wang et al., 2005). A new technique has been successfully developed to measure insitu temperature and oxygen concentration inside LTCC goafs. This paper describes the technique and its application.

FIELD TESTS AND RESULTS

Test Site

A field test was undertaken in #4326 LTCC face of Xinglongzhuang coal mine of Yankuang Group, China. The #3 coal seam is mined with an average thickness of 8.6 m (mining height is 3 m and caving height is 5.6 m) and the seam dips at 6°. The panel length is 1410 m and the face is 300 m wide. The overburden depth ranges from 470 m to 517 m. The seam is prone to sponcom with an incubation period of 3-6 months (the shortest incubation period is only 22 days). The face is "U" type ventilated with an air flow of 20 m³/s. A schematic of the panel layout is shown in Figure 1.

Figure 1 - Layout of #4326 LTCC face, Xinglongzhuang mine

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² CSIRO Exploration & Mining, PO Box 883, Kenmore QLD, Australia 4069.
Test Technique

A total of 7 measurement points were placed behind the rear AFC along the face using a 50m spacing and numbered as #1, #2, #3, #4, #5, #6 and #7 respectively (Figure 2).

![Figure 2 - Layout of measurement points at #4326 LTCC face](image)

At each measurement point, a temperature sensor and gas sampling tube were installed, and the sensor and tube were contained inside a perforated short steel pipe. A quick connector is fitted in the pipe for a signal transfer wire and the tube to be connected with a connector in a long steel pipe installed along the face (Figure 3). This enables the continuous and simultaneous temperature measurements and gas sampling at the 7 measurement points. As the face retreats, the perforated steel pipes containing sensors and tubes are buried inside the goaf, and the temperature and gas concentration inside the goaf are then measured.

![Figure 3 - Schematic of a measurement point](image)

1 tube; 2 temperature wire; 3 temperature sensor; 4 opening in steel pipe; 5 dust filter; 6 steel pipe; 7 sealing material; 8 quick connector

Test Results and Discussions

The field test lasted for 30 days while the face retreated over 240 m. Test results are shown in Table 1. The results were then analysed in terms of the variation of temperature and oxygen concentration with the face retreat and are shown in Figures 4 and 5.
Figure 4 reveals that the temperature inside the goaf increases with face retreat, though the temperature rise is fairly moderate (about 4°C within one month over 200 m). The highest temperature rise occurred at #1 measurement point, i.e. inside the goaf along the intake gateroad, with an average temperature increase of 0.19°C/d. In the whole test period, there was no occurrence of 1°C/d, indicating the sponcom prone zone in this goaf cannot be determined with temperature measurements alone.

Figure 5 shows the distribution of oxygen concentration inside #4326 goaf. The oxygen concentration reflects the ventilation air flow velocity and seam gas accumulation which are related to strata re-consolidation. In terms of the sponcom risk inside a goaf, the goaf area behind a longwall face is often divided into three zones, namely high (air flow) velocity zone (low sponcom risk), critical velocity zone

<table>
<thead>
<tr>
<th>Distance inside goaf (m)</th>
<th>#1 O₂</th>
<th>#2 O₂</th>
<th>#3 O₂</th>
<th>#4 O₂</th>
<th>#5 O₂</th>
<th>#6 O₂</th>
<th>#7 O₂</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>°C</td>
<td>%</td>
<td>°C</td>
<td>%</td>
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<tr>
<td>15</td>
<td>20.7</td>
<td>25.6</td>
<td>17.7</td>
<td>25.6</td>
<td>17.6</td>
<td>25.7</td>
<td>17.1</td>
</tr>
<tr>
<td>20</td>
<td>20.4</td>
<td>26.1</td>
<td>17.4</td>
<td>26.2</td>
<td>16.8</td>
<td>26.2</td>
<td>16.5</td>
</tr>
<tr>
<td>25</td>
<td>18.9</td>
<td>26.4</td>
<td>16.1</td>
<td>26.3</td>
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<td>30</td>
<td>18.4</td>
<td>26.7</td>
<td>15.7</td>
<td>26.8</td>
<td>15.2</td>
<td>26.9</td>
<td>14.7</td>
</tr>
<tr>
<td>35</td>
<td>17.6</td>
<td>26.9</td>
<td>15.5</td>
<td>26.9</td>
<td>14.5</td>
<td>26.9</td>
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<td>40</td>
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<td>13.6</td>
<td>27.4</td>
<td>13.6</td>
<td>27.4</td>
<td>11.9</td>
</tr>
<tr>
<td>50</td>
<td>15.8</td>
<td>27.6</td>
<td>12.1</td>
<td>27.5</td>
<td>11.9</td>
<td>27.8</td>
<td>11.5</td>
</tr>
<tr>
<td>55</td>
<td>15.8</td>
<td>27.7</td>
<td>11.7</td>
<td>27.6</td>
<td>11.1</td>
<td>27.9</td>
<td>10.7</td>
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<tr>
<td>75</td>
<td>14.6</td>
<td>27.9</td>
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<td>9.32</td>
<td>27.9</td>
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</tr>
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<td>100</td>
<td>14.3</td>
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<td>6.93</td>
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<td>150</td>
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<td>29.6</td>
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<td>29.4</td>
<td>5.92</td>
<td>29.0</td>
<td>5.89</td>
</tr>
<tr>
<td>200</td>
<td>6.2</td>
<td>29.1</td>
<td>6.15</td>
<td>29.2</td>
<td>5.95</td>
<td>29.3</td>
<td>5.74</td>
</tr>
</tbody>
</table>

Table 1 - Measured temperature and oxygen concentration in #4326 LTCC goaf

![Figure 4 - Temperature variations inside #4326 LTCC goaf with face retreat](image)
Figure 5 - Contour of oxygen concentration inside #4326 LTCC goaf

(high sponcom risk) and low (air flow) velocity zone (low sponcom risk). In the case #4326 goaf, the three zones were derived from Figure 5 and shown in Table 2.

Table 2 - Three zones inside #4326 LTCC goaf

<table>
<thead>
<tr>
<th>Zone</th>
<th>high velocity</th>
<th>critical velocity</th>
<th>low velocity</th>
<th>width</th>
</tr>
</thead>
<tbody>
<tr>
<td>goaf along intake gateroad</td>
<td>0-35.3 m</td>
<td>35.3-160 m</td>
<td>&gt;160 m</td>
<td>10-20 m</td>
</tr>
<tr>
<td>goaf behind face</td>
<td>0-14.6 m</td>
<td>14.6-75 m</td>
<td>&gt;75 m</td>
<td>260-280 m</td>
</tr>
<tr>
<td>goaf along return gateroad</td>
<td>0-28 m</td>
<td>28-100 m</td>
<td>&gt;100 m</td>
<td>10-16 m</td>
</tr>
</tbody>
</table>

Results from Table 2 indicate that the high sponcom risk zone in #4326 LTCC face is quite extensive. The goaf zone along the intake gateroad is quite long (35.3-160 m range). This is because of the high permeability in this area due to pillar support and the continuous fresh air feed from the intake. The goaf zone along the return gateroad is relatively shorter in comparison with that along the intake gateroad (28-100 m range) because of relatively low oxygen concentration from a small amount of air leakage into the area. The high sponcom risk goaf zone in the middle of the face is restricted to the area 15-75 m behind the face.

The identification of the high sponcom risk zone from the test can be used to manage sponcom risk. For example, it can be used to calculate the minimum mining rate by taking account of the incubation period of the seam and the extent of the zone. In the case of #4326 LTCC face, the minimum monthly mining rate for minimising sponcom risk was calculated as 170 m.

CONCLUSIONS

The main conclusions drawn from the field test are summarised as follows:

- A new technique has been developed for directly measuring the temperature and oxygen concentration inside a LTCC goaf, and the technique has been successfully applied in #4326 LTCC face of Xinglongzhuang mine.
- Results from the measurements can be used to identify the sponcom risk zone inside the goaf.
- Locating the high sponcom risk zones inside an active goaf should rely on oxygen concentration.
distribution, with temperature measurements as an auxiliary method.

- In the high sponcom risk zone in a LTCC face, the goaf areas along the intake and return gateroads are longer than that along the middle of the face.

REFERENCES


GAS DRAINAGE PRACTICES AND CHALLENGES IN COAL MINES OF CHINA

Kai Wang\textsuperscript{1,2}, and Sheng Xue\textsuperscript{2}

ABSTRACT: A number of gas drainage techniques are developed and practiced in many coal mines of China mainly to minimize outburst risk and reduce gas emission. Dependent upon local geological and mining conditions, one or more techniques may be practiced in a coal mine. A detailed review of the gas drainage practices and challenges in coal mines of China is presented, with particular reference to gas drainage techniques applicable to coal seams of low permeability.

OVERVIEW OF GAS DRAINAGE IN COAL MINES OF CHINA

China is the biggest coal-producing country in the world and coal output reached 2,100 Mt in 2005. Coal production from underground mines contributes 95\% of the total output and over 50\% of underground coal mines are classified as gassy and/or outburst prone (Fu, 2005). Chinese coal mines have a high rate of accidents, among the incidents, gas-related disasters account for over 40\%, and 82\% of major incidents (over 10 fatalities in a single incident) are caused by gas explosion (Yuan, 2004).

Gas drainage is the most effective measure for mine gas control. By 2002, gas drainage systems were set up in 193 mines in China and total volume of gas drained reached 1.1461 billion m\textsuperscript{3}. In 2002, average mine-wide gas drainage ratio was 26.6\%, average panel-wide gas drainage ratio was 37.6\%, average methane concentration of drained gas was 30.3\%, and there were 35 coal mines with the amount of gas drainage exceeding 10 Mm\textsuperscript{3}. Table 1 lists the volume of drained gas, ratio of gas drainage, and utilizing ratio of drained gas of these 35 mines (Fu, 2005; Wang, 2003).

GAS DRAINAGE TECHNIQUES

Seam classification in terms of ease of gas drainage

Based on ease of seam gas drainage, seams are classified into three categories, namely: easily drainable, drainable and hardly drainable. The classification is quantified by the decay rate of gas flow and seam permeability, as shown in Table 2 (Yu, 1992). For seams classified as drainable and easily drainable, conventional inseam gas drainage is practiced; for seams in hardly drainable category, inter-crossing inseam gas drainage and/or drainage after distressing measures are taken is practiced.

<table>
<thead>
<tr>
<th>No</th>
<th>Mine</th>
<th>Company</th>
<th>Drainage volume Mm\textsuperscript{3}</th>
<th>Drainage ratio %</th>
<th>Utilizing ratio %</th>
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<tr>
<td>1</td>
<td>Laohtai</td>
<td>Fushun</td>
<td>127.60</td>
<td>81.7</td>
<td>93</td>
</tr>
<tr>
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<td>No.5 Yangquan</td>
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<td>90.29</td>
<td>79.9</td>
<td>10</td>
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<tr>
<td>3</td>
<td>Houpun</td>
<td>Qinshui</td>
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<td>54.6</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>No.2 Yangquan</td>
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<td>33.92</td>
<td>29.5</td>
<td>90</td>
</tr>
<tr>
<td>5</td>
<td>No.1 Yangquan</td>
<td></td>
<td>33.82</td>
<td>47.9</td>
<td>64</td>
</tr>
<tr>
<td>6</td>
<td>Bajigou</td>
<td>Ningxia</td>
<td>30.80</td>
<td>53.0</td>
<td>100</td>
</tr>
<tr>
<td>7</td>
<td>Datong No.1</td>
<td>Songzao</td>
<td>27.66</td>
<td>43.5</td>
<td>95</td>
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<tr>
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<td>Panji No.1</td>
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<td>13</td>
<td>Tucheng</td>
<td>Panjiang</td>
<td>20.28</td>
<td>55.0</td>
<td>-</td>
</tr>
</tbody>
</table>

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\textsuperscript{2} CSIRO Exploration & Mining, PO Box 883, Kenmore Queensland
2008 Coal Operators’ Conference

The AusIMM Illawarra Branch

14 Datong No.2 Songzao 18.96 48.0 90
15 No.3 Yangquan 18.07 24.0 93
16 Huopu Panjiang 17.39 30.1 -
17 Songzao Songzao 16.79 42.2 85
18 Daxing Tiefa 16.53 39.7 41
19 Panji No.3 Huainan 15.70 36.0 11
20 Sihe Jincheng 15.00 60.0 -
21 Hongling Shengyang 14.64 41.3 21
22 Dalong Tiefa 14.64 40.6 51
23 Dawan Shuicheng 14.30 39.5 5
24 South Zhongliangsha 14.24 56.0 100
25 Shihao Songzao 13.30 41.5 98
26 Laowuji Panjiang 12.68 44.7 -
27 Wangjiazai Panjiang 12.66 34.2 13
28 Xinzhuangzi Huainan 11.57 22.3 -
29 Nanshan Hegang 11.39 35.0 66
30 Wulong Fuxin 11.36 32.0 -
31 Moxinpoo Tianfu 11.06 44.1 97
32 Yonghong Qinshui 11.01 22.7 -
33 Muchonggou Shuicheng 10.64 44.4 -
34 Xiaoming Tiefa 10.34 45.5 45
35 Yueliangtian Panjiang 10.33 30.1 9

Total 83,544
Average 44.7 51

Table 2 - Seam classifications in terms of gas drainage ease

<table>
<thead>
<tr>
<th>Category</th>
<th>Decay coefficient of gas flow from a inseam borehole of 100m in length, d⁻¹</th>
<th>Seam permeability* m⁻²MPa⁻²d⁻¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Easily drainable</td>
<td>&lt; 0.005</td>
<td>&gt;10</td>
</tr>
<tr>
<td>Drainable</td>
<td>0.005–0.05</td>
<td>10–0.1</td>
</tr>
<tr>
<td>Hardly drainable</td>
<td>&gt; 0.05</td>
<td>&lt;0.1</td>
</tr>
</tbody>
</table>

* 1 m⁻²MPa⁻²d⁻¹ is equivalent to 0.025 md

Gas drainage techniques and their applicable seam conditions

Based upon gas sources, gas drainage techniques are divided into working seam drainage, adjacent seam drainage, and goaf drainage. In terms of where gas flows through, the techniques are divided into borehole and tunnel techniques. Gas drainage techniques are also classified as drainage with and without de-stressing. Gas drainage can also be divided into underground drainage and surface drainage. Various gas drainage techniques are practiced in coal mines of China, and their applicable conditions are summarized in Table 3. Selection of appropriate gas drainage technique(s) for a coal mine depends mainly on site specific geological and mining conditions, such as seam permeability, seam gas content, seam hardness, sources of gas emission, as well as cost.

Gas drainage practice

Working seam drainage

Most coal mines extracting a single gassy or outburst prone seam adopt the technique of gas pre-drainage prior to mining, such as the mines in Jiaozuo, Hebi, Jincheng and Lu’an mining areas. Some mines which extract multiple seams also use this technique to drain gas in protective seams. Sometimes in order to overcome the problem of insufficient gas drainage lead time and increase drainage ratio, techniques of gas drainage while mining are also applied. To minimize outburst risk and control high gas emission during seam roadway development, some mines adopt the techniques of gas drainage while developing seam roadways.
Adjacent seam drainage

If gas emission at a mining face mainly comes from de-stressed adjacent seams and face ventilation circuit couldn’t provide sufficient air quantity to dilute the high gas emission, then techniques of adjacent seam drainage are applied. In this case, most faces (70%) use cross-measure boreholes to drain gas from adjacent seams. Such technique is widely used in Yangquan, Tianfu, Songzao and Zhongliangshan mining areas, and the drainage ratio of these faces is usually over 50%.

Technique of specially extracted gas drainage tunnel in an adjacent seam has been successfully tried in Yangquan No.1 mine (Bao et al., 1996). This technique is also been named as high position drainage tunnel which is capable of draining more gas than conventional boreholes. The drainage ratio of upper adjacent seam with this technique can reach up to 85%, which is suitable to mining faces where gas emission from upper adjacent seam is over 30 m³/min.

It is well known that gas in adjacent seams can be effectively drained if the seams lie in a fractured zone (de-stressed). Recent practice in Huainan mining area indicates that if adjacent seams lie in a deformed and subsided zone, gas from the seam can also be effectively drained with high efficiency (Yu et al., 2004). Technique of adjacent seam drainage has been widely applied in many mining areas with satisfactory results.

Table 3 - Gas drainage techniques and their applicable conditions

<table>
<thead>
<tr>
<th>Technique of gas drainage</th>
<th>Applicable conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-drainage of seam roadway development  (without de-stressing)</td>
<td>Cross-measure boreholes drilled from rock roadway</td>
</tr>
<tr>
<td>Inseam boreholes ahead of inseam roadway development</td>
<td>Outburst prone seam Gassy seam</td>
</tr>
<tr>
<td>Pre-drainage of working face</td>
<td>Inseam boreholes</td>
</tr>
<tr>
<td>Drainage without de-stressing</td>
<td>Outburst prone seam Gassy seam</td>
</tr>
<tr>
<td>Drainage of working face</td>
<td>Cross-measure seam boreholes drilled from cross measure</td>
</tr>
<tr>
<td>roadway, rock roadway or roadway in adjacent seams</td>
<td>Drainable seam</td>
</tr>
<tr>
<td>Surface boreholes</td>
<td>Outburst prone seam Gassy seam</td>
</tr>
<tr>
<td>Drainage while developing seam roadway</td>
<td>Gassy and easily drainable seam</td>
</tr>
<tr>
<td>Drainage while face retreating /advancing</td>
<td>Relatively shallow seam</td>
</tr>
<tr>
<td>Inter-crossing pre-drainage (measures taken to increase</td>
<td>Inseam boreholes</td>
</tr>
<tr>
<td>increase seam permeability)</td>
<td>Boreholes drilled from rock roadway or surface</td>
</tr>
<tr>
<td>Drainage of working face</td>
<td>Gassy and hardly drainable seam</td>
</tr>
<tr>
<td>Drainage with de-stressing</td>
<td>High gas emission from adjacent seam.</td>
</tr>
<tr>
<td>Drainage of overlying and underlying seams</td>
<td></td>
</tr>
<tr>
<td>Drainage of overlying and underlying seams</td>
<td>Cross-measure boreholes to adjacent seam</td>
</tr>
<tr>
<td>Drainage of overlying and underlying seams</td>
<td>High gas emission from adjacent seam and normal boreholes</td>
</tr>
<tr>
<td>Tunnel in adjacent seam for gas drainage</td>
<td>cannot cope with gas emission</td>
</tr>
<tr>
<td>Surface boreholes</td>
<td>When surface borehole is considered to be a better</td>
</tr>
<tr>
<td>Adjacent seam drainage with de-stressing</td>
<td>option than underground borehole.</td>
</tr>
<tr>
<td>Surface boreholes</td>
<td>No sponcom risk seam</td>
</tr>
<tr>
<td>Goaf drainage</td>
<td>Sponcom risk seam where measures taken to mitigate sponcom</td>
</tr>
<tr>
<td>Pipes placed in goaf</td>
<td>risk are considered safer than underground borehole.</td>
</tr>
<tr>
<td>Boreholes into goaf</td>
<td></td>
</tr>
<tr>
<td>Surface boreholes</td>
<td></td>
</tr>
</tbody>
</table>
**Goaf drainage**

In cases where gas emission into a mining face is mainly from goaf, the techniques of goaf drainage are usually adopted. These include cross-measure boreholes into roof strata, placing gas drainage pipes in goaf, and surface boreholes.

Cross-measure boreholes into fractured roof strata, as shown in Figure 1, have been proven to be quite effective. The boreholes are drilled from panel return side into fractured roof strata at an upside angle of $10^\circ-18^\circ$, away from the return at an angle of $15^\circ-20^\circ$, and 80-140 m in length. Two adjacent drilling insets are spaced 50-80 m. At each inset, 3 to 5 boreholes are drilled, and this leads to borehole overlapping around 40-65 m. With the layout of boreholes, amount of gas drained from the boreholes can be kept fairly constant when the face is mined through the inset. Borehole diameter varies from 50 mm to 127 mm, the larger a borehole diameter, the higher gas flow rate from the borehole. Field observations from Daxing mine in Tiefa mining area show that when borehole diameter is 50 mm, 75mm, 89 mm, 108 mm and 127 mm, the respective gas flow rate from the borehole is 0.3-0.5, 1.5-2.0, 3.0-4.0, 5.0-7.0 and 7.0-8.0 m$^3$/min.

Placing gas drainage pipes in goaf is another technique in use in coal mines of China. In order to ensure a certain quantity of gas drainage, pipe diameter should not be less than 150 mm, and the pipe made of magnesite are used to reduce cost.

Surface borehole goaf drainage has been trialed in some mines in China with mixed results. If coal seams are more than 600 m below the surface, its application may be also complicated with borehole stability.

![Figure 1 - Goaf gas drainage with cross-measure boreholes into fractured roof strata](image)

**Challenges**

At present the main problem of gas drainage is low drainage ratio. The low drainage ratio is mainly caused by lack of effective pre-drainage technique for seams of low permeability and poor management of gas drainage practice. Nearly 95% of gassy and outburst prone mines in China are mining seams of $10^{-3}-10^{-4}$ md permeability, and conventional gas pre-drainage is ineffective. Poor management of gas drainage practice include inadequate drainage lead time, insufficient number of boreholes, poor sealing of boreholes, lack of gas drainage monitoring system, and inappropriate gas drainage system (Wang, 2003).

**GAS DRAINAGE IN SEAMS OF LOW PERMEABILITY**

Targeted at highly gassy and outburst prone seams of low permeability, a number of technologies have been developed to enhance gas drainage over the last 30 years in China. These technologies include hydraulic or high pressure air fracturing or cracking of seams, high pressure water injection for borehole enlarging, blasting for coal loosening, controlled blasting in long boreholes for coal pre-fracturing, and inter-crossing boreholes drainage (cross-measure boreholes, inseam boreholes, large diameter boreholes) (Fu, 2005; Yu, 1992; Bao et al., 1996; Wang, 1992; Wang, 2002). Among these technologies, inter-crossing borehole drainage, hydraulic coal cracking and controlled blasting in long boreholes for coal pre-fracturing are proven to be more effective and easy to implement because of simple equipments requirements.
Inter-crossing borehole drainage

In inter-crossing boreholes drainage, two groups of boreholes are drilled to increase the intensity of gas drainage. One group of boreholes is drilled in parallel and the other group intercrossing over or below the former, as shown in Figure 2.

Results of inseam gas drainage with parallel boreholes alone and inter-crossing boreholes at some sites are shown in Table 4. Results shown in Table 4 revealed that at the same site and with the same drainage lead time:

- amount of gas drained with large diameter (150 mm or 300 mm) parallel boreholes was 2.5 times more than that with normal diameter (65-75 mm).
- amount of gas drained with intercrossing boreholes is 1.5 - 2.0 times more than that with parallel boreholes alone with the same borehole intensity and diameter.

### Table 4 - Results of gas drainage by parallel boreholes and intercrossing boreholes

<table>
<thead>
<tr>
<th>Technique</th>
<th>Site</th>
<th>Seam parameters</th>
<th>Borehole parameter s</th>
<th>Gas flow from 100m long borehole (q=0.05\text{m}^3\text{min}^{-1}\text{hm}^{-1})</th>
<th>Drainage Quantity, 10^{-5} m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel boreholes</td>
<td>No.912 face of Yangquan No.1 mine</td>
<td>Thickness, m</td>
<td>2.2</td>
<td>Spacing, m</td>
<td>Initial flow rate, q₀</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dip, °</td>
<td>&lt;10</td>
<td>18.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Large diameter parallel</td>
<td></td>
<td>Gas content, m³⁻¹</td>
<td></td>
<td></td>
<td>Permeability, m² MPa⁻²d⁻¹</td>
</tr>
<tr>
<td>boreholes</td>
<td></td>
<td>Gas pressure, MPa</td>
<td></td>
<td></td>
<td>Spacing, m</td>
</tr>
<tr>
<td></td>
<td>Large diameter parallel</td>
<td>Diameter, mm</td>
<td>73</td>
<td></td>
<td>17.0</td>
</tr>
<tr>
<td>boreholes</td>
<td></td>
<td>Spacing, m</td>
<td>1.7</td>
<td></td>
<td>18.3</td>
</tr>
<tr>
<td></td>
<td>No.42081 and 41041 faces of Jiaoxi</td>
<td>Diameter, mm</td>
<td>75</td>
<td></td>
<td>17.7</td>
</tr>
<tr>
<td>boreholes</td>
<td>mine</td>
<td>Spacing, m</td>
<td>1.3</td>
<td></td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permeability, m²</td>
<td>0.55</td>
<td></td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MPa⁻²d⁻¹</td>
<td></td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial flow rate, q₀</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Decay coefficient B, -¹</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drainage Quantity, 10^{-5} m³</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel boreholes</td>
<td>No.13501 face of Jiaozhou Jiulishan</td>
<td>Diameter, mm</td>
<td>65</td>
<td></td>
<td>16.9</td>
</tr>
<tr>
<td>boreholes</td>
<td>mine</td>
<td>Spacing, m</td>
<td>2.4</td>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permeability, m²</td>
<td>3.5</td>
<td></td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MPa⁻²d⁻¹</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial flow rate, q₀</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drainage Quantity, 10^{-5} m³</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercrossing boreholes</td>
<td>No.13501 face of Jiaozhou Jiulishan</td>
<td>Diameter, mm</td>
<td>65</td>
<td></td>
<td>16.9</td>
</tr>
<tr>
<td></td>
<td>mine</td>
<td>Spacing, m</td>
<td>2.4</td>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permeability, m²</td>
<td>3.5</td>
<td></td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MPa⁻²d⁻¹</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial flow rate, q₀</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drainage Quantity, 10^{-5} m³</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel boreholes</td>
<td>E10-20100 face of Pingdingshan No.10</td>
<td>Diameter, mm</td>
<td>4.2</td>
<td></td>
<td>13.5</td>
</tr>
<tr>
<td>boreholes</td>
<td>mine</td>
<td>Spacing, m</td>
<td>2.0</td>
<td></td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permeability, m²</td>
<td>2.0</td>
<td></td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MPa⁻²d⁻¹</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial flow rate, q₀</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drainage Quantity, 10^{-5} m³</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2 - Inter-crossing boreholes for inseam gas pre-drainage
Hydraulic coal cracking and controlled blasting in long boreholes

Controlled blasting in long boreholes aims to enhance seam permeability through coal pre-fracturing. To control fracturing direction and increase free surface area, not all boreholes are filled with explosives and blasted, instead there are some boreholes left deliberately between two charged boreholes. Hydraulic coal cracking aims to increase seam permeability through cutting two 0.3-0.6 m wide cracks at both sides of an inseam borehole by high pressure water injection.

Results of hydraulic coal cracking and controlled blasting in long boreholes at some sites are shown in Table 5. Results shown in Table 5 indicated that:

- Seam permeability was increased by 2 to 5 times by controlled blasting in long boreholes, and amount of gas drained after blasting was increased by 50 - 90%.
- Seam permeability was increased by 10 to 100 times by hydraulic coal cracking, and amount of gas drained after cracking was increased by 100-200%.

Discussion

By taking into considerations of equipment requirements, maturity of technology, effectiveness, practicality, operational safety and cost, the most feasible gas drainage techniques in seams of low permeability are inter-crossing boreholes and intensive parallel boreholes of large diameter. If underground conditions are suitable, hydraulic coal cracking technique can significantly reduce drilling operations and the technique can be used to replace intensive parallel boreholes. Technique of controlled blasting in long boreholes has the similar effect to that of inter-crossing boreholes, although its technical requirement is stricter because of drilling difficulty and operation of placing explosive charge in boreholes and blasting. As the borehole becomes longer, chance of successful blasting decreases, and safety risk of whole operation increases.

Technique of hydraulic coal fracturing/cracking requires specially complex and heavy equipment, and there are still issues to be resolved as how to control inseam fractures and what kind of materials are more appropriate to support the fractures. Furthermore effectiveness of the technique has not yet widely demonstrated.

MANAGEMENT STRATEGY TO INCREASE MINE GAS DRAINAGE RATIO

Increasing borehole length

Borehole length is an important factor affecting gas drainage. In outburst prone seams of low permeability, drilling of long boreholes is difficult because the boreholes can be badly deformed, bursting can occur while drilling, and flushing cuttings can be problematic. Therefore more advanced and effective drilling equipment suitable for long-hole inseam drilling should be developed as a matter of urgency. The drilling equipment should be directional, more powerful, and capable of removing cuttings and preventing bursting while drilling long inseam borehole.

Improving borehole sealing

Gas purity of inseam gas pre-drainage is a major issue. Of all working faces where inseam gas pre-drainage is practiced in China, about 65 % of them drain gas with its purity below 30%. One reason lies with borehole sealing, including sealing materials and sealing length. Clay and slurry of cement and sand are used to seal gas drainage boreholes in about 2/3 of mines. Recent practice indicates that polyurethane has high expansive coefficient, short coagulating duration, high sealing efficiency, small shrinkage and good sealing quality. It has been used in some mines to seal gas drainage borehole with good results. Sealing length is normally 4 – 6 m, and it needs to be increased.

Optimizing drainage system

Optimization of gas drainage system may include: (1) selection of suitable pumps to match gas volume and resistance of drainage reticulation system; (2) increasing the diameter of gas pipes; (3) installing automatic devices to discharge water in drainage reticulation system; and (4) regularly conducting leak check and maintenance of drainage system.

Increasing gas drainage lead time

For seams of low permeability (less than 10^{-3} md), drainage lead time must be over 6-8 months to realize moderate drainage ratio. In China, roadway development rate in outburst prone seams is usually less than 100 m per month, and schedule of roadway development and face retracting is fairly tight, which leaves little lead time for gas drainage. Data from Jiaozuo, Hebi, Pingdingshan, Huainan, Huaibei, Fushun, Tiefa mining areas reveals that average gas drainage lead time in outburst prone seams is only 3-4 months. It is therefore necessary to optimize
mine planning to ensure sufficient gas drainage lead time, and at the same time to develop techniques to rapidly minimize outburst risk.

Table 5 - Results of gas drainage by controlled blasting in long boreholes and hydraulic coal cracking

<table>
<thead>
<tr>
<th>Technique</th>
<th>Site</th>
<th>Seam parameters</th>
<th>Gas flow from 100m long borehole, ( q = q_0 e^{-Bt} )</th>
<th>100m long borehole, ( q_0 ) (m³/min hm⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Seam thickness, (m)/Dip, (°)</td>
<td>Permeability, (m² MPa⁻² d⁻¹)/Before treatment</td>
<td>Explosive, (kg)/Length of explosive, (m)</td>
</tr>
<tr>
<td>Parallel boreholes</td>
<td>No.41041 face of Jiaoxi mine</td>
<td>5.3/10&lt;10</td>
<td>2.08/2.08</td>
<td>40/30</td>
</tr>
<tr>
<td>Controlled blasting</td>
<td></td>
<td>5.3/10&lt;10</td>
<td>2.08/7.18</td>
<td>12/5</td>
</tr>
<tr>
<td>Parallel boreholes</td>
<td>F17-2228 face of Pingdingshan No.5</td>
<td>5.9/10&lt;10</td>
<td>0.504/0.504</td>
<td>191/256/49/71</td>
</tr>
<tr>
<td>Controlled blasting</td>
<td></td>
<td>5.9/10&lt;10</td>
<td>0.504/3.03</td>
<td>191/256/49/71</td>
</tr>
<tr>
<td>Hybrid coal cracking</td>
<td>Hebi No.4 mine</td>
<td>4.5-9/10&lt;10</td>
<td>14.6-15.1/1.9</td>
<td>0.04/0.36-0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5-9/10&lt;10</td>
<td>14.6-15.1/1.9</td>
<td>0.04/0.36-0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.5/30</td>
<td>14.3/1.6</td>
<td>0.24-0.4/9.8-55.2</td>
</tr>
</tbody>
</table>

**OUTLOOK**

As coal seams become deep and coal production increases, gas emission will increase and gas-related risk will also increase. Moreover seam permeability will decrease further as seam overburden increases, and it will be more difficult to drill boreholes and effectively pre-drain gas. Therefore the following tasks should be targeted:

- A combination of gas drainage techniques must be applied to tackle multiple sources of gas emission at working face, the techniques include working seam drainage, adjacent seam drainage, and goaf gas drainage.
- Gas drainage technology should be further developed towards drilling large diameter (>150 mm), directional and long (200-500 m) intercrossing boreholes. Technologies to enhance seam permeability should also be further developed in terms of their effectiveness and practicability.
- De-stressed gas drainage techniques should be further developed with detailed understanding of mining-induced de-stressed zone(s).

In summary, development of effective gas drainage technology is vital to realize safe extraction of coal and gas and ensure sustainable development of coal mines in China.
ACKNOWLEDGEMENT

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REFERENCES

**IMPROVING UIS GAS DRAINAGE IN UNDERGROUND COAL MINES**

**Dennis J. Black** and **Naj I. Aziz**

**ABSTRACT:** Gas drainage is considered one of the most effective controls in preventing coal and gas outbursts. The advances in mining equipment technology over the last 20 years have led to a significant increase in coal mine production, resulting in increased coal mine gas emission during the coal extraction process. High gas emissions, if not effectively managed, may exceed the diluting capability of the mine’s ventilation system, potentially exceeding the statutory limit, resulting in gas related production delays. The development of underground to inseam (UIS) gas drainage is discussed. The study is focused on a range of factors which impact gas drainage and proposes actions to improve drainage performance.

**INTRODUCTION**

Many Australian underground coal mines are progressing toward areas which require the use of gas drainage to reduce seam gas concentrations to below a prescribed Threshold Limit Value (TLV). In a number of cases, these mines will encounter areas where the gas is extremely difficult to drain from the coal, ahead of mining. Where difficult drainage areas are encountered, the mines may be faced with significant production delays while intensive drilling is carried out to reduce the gas concentrations to acceptable levels. To avoid such costly delays, mine management may choose to avoid the area completely, resulting in loss of reserves, loss of potential revenue and ultimately reduced mine life.

This project aims, through an integrated series of site based and laboratory studies to determine the relative impact and significance of a broad range of ‘factors’ which impact gas drainage in underground mines. A further aspect of this study involves the assessment of a range of actions, based on knowledge of the factors identified in the early stage of the project, which can be taken by the mine operator to improve gas drainage performance, enabling the potentially sterilised areas to be accessed and extracted.

**BACKGROUND**

This study focuses on gas drainage experience in the Bulli seam which is located in the southern Sydney Basin in New South Wales, Australia. The seam is stratigraphically the uppermost coal seam in the Permian Illawarra Coal Measures. The depth of cover, in the area of study is in the order of 450 to 500 metres with a regional dip of approximately 1.9 degrees, toward the west. The gas composition in the mining domain is variable, ranging from almost pure CH₄ in the east to almost pure CO₂ in the west. Table 1 contains details of the range of coal properties investigated, which are characteristic of the mining area investigated, known as Mine A.

<table>
<thead>
<tr>
<th>Coal Property</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH₄ ratio (CH₄/CH₄+CO₂)</td>
<td>6.5%</td>
<td>97.8%</td>
</tr>
<tr>
<td>Gas Content (m³/t)</td>
<td>5.2 m³/t</td>
<td>15.72 m³/t</td>
</tr>
<tr>
<td>Permeability (mD)</td>
<td>0.05 mD</td>
<td>6 mD</td>
</tr>
<tr>
<td>Seam thickness (m)</td>
<td>2.33 m</td>
<td>3.19 m</td>
</tr>
<tr>
<td>Ash content (%)</td>
<td>9.4 %</td>
<td>14.8 %</td>
</tr>
<tr>
<td>Moisture content (%)</td>
<td>0.7 %</td>
<td>1.3%</td>
</tr>
<tr>
<td>Vitrinite reflectance (R_max)</td>
<td>1.23</td>
<td>1.36</td>
</tr>
<tr>
<td>Vitrinite (%)</td>
<td>28.9%</td>
<td>54.5%</td>
</tr>
<tr>
<td>Inertinite (%)</td>
<td>45.51%</td>
<td>71.1%</td>
</tr>
<tr>
<td>Mineral matter</td>
<td>1.8%</td>
<td>5.78%</td>
</tr>
<tr>
<td>Volatile matter</td>
<td>19.7%</td>
<td>25.25%</td>
</tr>
</tbody>
</table>

The Bulli seam is extremely prone to the occurrence of coal and gas outbursts. During the history of mining in the Bulli seam there have been 12 reported fatalities associated with outbursts, listed in Table 2.

---

1 Department of Civil, Mining and Environmental Engineering, University of Wollongong, Australia
Table 2 - Fatal outbursts in the Bulli seam (source, Harvey and Singh, 1998)

<table>
<thead>
<tr>
<th>COLLERY</th>
<th>DATE</th>
<th>KILLED</th>
<th>SIZE (tonnes)</th>
<th>GAS</th>
<th>STRUCTURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metropolitan</td>
<td>10 June 1896</td>
<td>3</td>
<td>Unknown</td>
<td>CH4</td>
<td>Dyke and soft fault</td>
</tr>
<tr>
<td>Metropolitan</td>
<td>27 July 1926</td>
<td>2</td>
<td>140</td>
<td>CO2</td>
<td>Fault with 5m throw</td>
</tr>
<tr>
<td>Metropolitan</td>
<td>2 December 1954</td>
<td>2</td>
<td>90</td>
<td>CO2</td>
<td>Normal fault with 0.3m throw</td>
</tr>
<tr>
<td>Tahmoor</td>
<td>24 June 1985</td>
<td>1</td>
<td>400</td>
<td>CO2</td>
<td>Dyke associated with strike-slip movement</td>
</tr>
<tr>
<td>South Bulli</td>
<td>25 July 1991</td>
<td>3</td>
<td>300</td>
<td>CO2 &amp; CH4</td>
<td>Thrust fault with 0.35m of mylonitic coal and very high gas pressure</td>
</tr>
<tr>
<td>West Cliff</td>
<td>25 January 1994</td>
<td>1</td>
<td>350</td>
<td>CO2</td>
<td>Intersection of 2 strike-slip structures and 0.3m of mylonitic coal</td>
</tr>
</tbody>
</table>

The most effective method of preventing an outburst is through the reduction of seam gas content and gas pressure, Lama (1980), Marshall, Lama and Tomlinson (1982) and Wood (1983). Over the years, many different approaches have been undertaken, by various mine operators, to reduce gas levels and pressures to manageable levels with mixed success.

However, the current advanced gas drainage technology is proving the most effective, Clark et al. (1983). Prior to 1980, gas drainage was achieved through drilling relatively short holes, typically 25 to 40 metres ahead of the working face, using primarily hand-held borers to drill holes of varying diameters, ranging from 43 mm to 100 mm, without any applied suction (Hargraves, 1983). Through advances in equipment technology increased coal production was achieved, however this also resulted in increased gas emissions into the mine airways. In order to prevent these gas emissions from exceeding the diluting capability of the mine’s ventilation system, regular, efficient gas drainage programs were required to reduce the gas content of the coal prior to mining, in addition to reducing the outburst risk.

In 1980, West Cliff Colliery commenced the first routine pre-drainage drilling and gas drainage program, ahead of mining. Since the commencement of underground to inseam (UIS) drilling the equipment has evolved from simple rotary drilling rigs with directional control accuracy of ±15° for hole lengths of 400–600 m (Hebblewhite et al., 1982, Hebblewhite et al., 1983 and Kelly 1983) to technically advanced units incorporating down-hole motors, with in-hole data acquisition tools, capable of drilling distances beyond 1,600 metres with survey accuracy in the order of ±0.5° azimuth and ±0.2° pitch (Valley Longwall Drilling).

Although aware of the relationship between gas and outburst risk, and the impact of gas drainage on reducing this risk, many operators failed to implement systems to regularly check and assess the outburst risk ahead of future workings.

Following the investigation into the last fatal outburst, which occurred in the Bulli seam at West Cliff Colliery on 25 January 1994, a directive was issued to all Bulli seam coal mine operators, under the authority of the Coal Mines Regulation Act 1982, prescribing threshold limit values (TLV), and other actions, to be implemented to manage risk and prevent future coal and gas outbursts. The Department of Mineral Resources, New South Wales, introduced The Outburst Mining Guideline, MDG 1004, in 1995, which led to the development and implementation of Outburst Management Plans, as part of the mine safety management system. Intensive UIS drilling programs are used to collect coal cores for gas content testing, identify structures ahead of the mine workings, as well as draining gas to below the TLV.

The outburst threshold limit graph which is typical of those introduced at each of the Bulli seam mines, and now forms an integral part of Outburst Management Plans (Figure 1), specifies maximum gas content to which the seam must be drained before approval may be given to mine the area. Given the high gas content throughout the Bulli seam, the introduction of the TLV graph has led to the development and implementation of intensive pre-drainage drilling programs aimed at reducing seam gas contents to below the applicable TLV, to avoid delays to the roadway development and longwall operations.
Many of the Bulli seam mines now employ extensive gas drainage programs with greater than 100,000 metres of underground to inseam (UIS) pre-drainage boreholes being drilled annually. However, even with the application of such intensive UIS drilling, there are a number of mining areas, past, present and future, which will experience difficulty reducing the gas content to below the TLV. This poor drainage is caused by a variety of geological conditions and operational factors which adversely impact drainage performance.

A number of inter-related projects are now being undertaken at the University of Wollongong to quantify the factors that impact gas drainage and to evaluate suitable drainage improvement initiatives that may be implemented by mine operators to address the poor drainage zones.

**Project Outline & Objectives**

The objective of this project is to develop an understanding of the factors which impact gas drainage and to determine suitable actions that may be implemented to successfully treat the identified difficult drainage zones, ahead of mining, enabling these zones to be accessed and extracted, without incurring any unacceptable gas drainage related production delays. The process to achieve this objective, summarised in Figure 2, involves both site-based and laboratory testing and analysis. The initial work programme focussed on determining the factors which have most impact on gas drainage, based on the collection and analysis of the following:

- gas flow data from UIS drill holes;
- coal core gas and quality data; and
- collect and test coal samples to determine properties and characteristics.

The second phase involves the investigation, evaluation and possible trial of a range of potential drainage improvement techniques, to improve gas drainage effectiveness.
Analyse gas emission from coal cores from current mining area to determine:
(a). the naturally retained gas content (Q3);
(b). the rate of gas emission; and
(c). changes in emitted gas composition through desorption.
The gas desorption results will be compared to results of petrographic analysis on the coal cores to determine relationships that may exist and impact gas drainage rate.

Detailed analysis of gas drainage experience in current mining domain determine:
(a). Factors and conditions that impact gas drainage performance;
(b). Opportunities to improve / optimise current underground to inseam (UIS) gas drainage program; and
(c). Identify areas where the ongoing use of UIS drilling alone will not achieve satisfactory drainage.

Test coal samples from a range of areas, representing different in situ conditions, and determine response to different gases.

Assess results and relate to the data obtained from site-based analysis of gas drainage experience and gas desorption analysis.

Identify and evaluate a range of alternate and stimulation options that may be employed by the mine to address areas of known difficult drainage.

Compile results of all analysis and investigation undertaken, summarising the opportunities that exist for the Colliery to improve the effectiveness of the current UIS gas drainage system, identify areas where the current drainage system will likely be ineffective and make recommendation regarding alternative method that may be employed to improve gas drainage performance.

Figure 2 - Project flowchart

Mine Site Analysis

Gas flow data has been collected from 306 UIS boreholes, covering an area which encompasses four separate longwall panels in a local mine (Mine A). The location of the boreholes and coal cores analysed during this study are shown in Figure 3.

A review of the entire dataset identified that 118 holes (38.5 % of the total holes) were compromised in some way, either through blockage or flooding, overlapping with adjacent boreholes, or very short life. These holes were excluded from the dataset as the flow data was deemed to be questionable and not representative of reasonable conditions. Gas flow data from the remaining 188 boreholes form the basis of the analysis. Table 3 lists the panels and number of boreholes from each which were included in the overall borehole flow analysis.
Figure 3 - UIS borehole and coal core locations analysed (Mine A)

Table 3 - Borehole number and location analysed in Mine A

<table>
<thead>
<tr>
<th>UIS Borehole Gas Flow Analysis Time on Suction</th>
<th>PANEL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>516</td>
</tr>
<tr>
<td>no.</td>
<td>%</td>
</tr>
<tr>
<td>Drill Stubs Recorded</td>
<td>3</td>
</tr>
<tr>
<td>Total holes in drill stub</td>
<td>33</td>
</tr>
<tr>
<td>No. holes &lt;100 days</td>
<td>14</td>
</tr>
<tr>
<td>No. holes &lt;50 days</td>
<td>8</td>
</tr>
<tr>
<td>Total holes included in analysis</td>
<td>13</td>
</tr>
</tbody>
</table>

Figure 4 shows the distribution of total gas production of all 188 boreholes included in the dataset.
Figure 4 - Distribution of total gas production from UIS boreholes

Table 4 lists the variables which were considered to have an impact on total gas production.

Table 4 - Factors initially considered to impact gas drainage

<table>
<thead>
<tr>
<th>Borehole length</th>
<th>Seam thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole orientation to cleat (100/280)</td>
<td>Moisture content</td>
</tr>
<tr>
<td>Borehole orientation to stress</td>
<td>Reflectance Rv(max)</td>
</tr>
<tr>
<td>Borehole apparent dip</td>
<td>Permeability (mD)</td>
</tr>
<tr>
<td>Gas content (m3/t)</td>
<td>Ash content</td>
</tr>
<tr>
<td>Gas composition (%CH4)</td>
<td>Intertinite component</td>
</tr>
<tr>
<td>Time on suction (days)</td>
<td>Vitrinite component</td>
</tr>
<tr>
<td>Suction pressure (kPa) - median</td>
<td>Mineral component</td>
</tr>
</tbody>
</table>

RESULTS AND DISCUSSION

A detailed statistical analysis was performed on the dataset from which the correlations, shown in Table 5, were determined. The table is split into the factors which are considered naturally occurring and therefore not controllable and those on the right which the operator has a certain degree of control over.

Table 5 - Statistical correlation results from complete dataset analysis

<table>
<thead>
<tr>
<th>Factor assessed for correlation</th>
<th>R²</th>
<th>Factor assessed for correlation</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas composition (%CH4)</td>
<td>42</td>
<td>Time on suction (days)</td>
<td>18</td>
</tr>
<tr>
<td>Gas content (m3/t)</td>
<td>32</td>
<td>Suction pressure (kPa) - median</td>
<td>10.2</td>
</tr>
<tr>
<td>Seam thickness</td>
<td>25</td>
<td>Borehole orientation to stress</td>
<td>6</td>
</tr>
<tr>
<td>Permeability (mD)</td>
<td>19</td>
<td>Borehole length</td>
<td>3.9</td>
</tr>
<tr>
<td>Reflectance Rv(max)</td>
<td>17</td>
<td>Borehole orientation to cleat (100/280)</td>
<td>1.1</td>
</tr>
<tr>
<td>Intertinite component</td>
<td>17</td>
<td>Borehole apparent dip</td>
<td>0</td>
</tr>
<tr>
<td>Vitrinite component</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mineral component</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ash content</td>
<td>6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Note: $R^2$ represents the percentage variation of the variable relative to the regression model for the dataset. The higher the $R^2$, the better the regression model fits the data.

Using statistical analysis, virtually no correlation was found between total gas production and the following variables; borehole orientation to cleat, borehole orientation to stress, and apparent dip, yet a review of the data distribution indicated ranges where greater gas production was achieved. Figure 5 shows the distribution of gas production data for these three variables.

The dataset was refined to include only those meeting all three of the criteria listed below. Further analysis was undertaken on the refined dataset to include:

- Borehole orientation to cleat (100/280) – 20 to 70 degrees;
- Borehole orientation to stress – 0 to 50 degrees; and
- Borehole apparent dip – 0 to +1.6 degrees.

Table 6 shows the results of the statistical analysis on the dataset which satisfied the above listed criteria.

<table>
<thead>
<tr>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas content (m³/t)</td>
<td>30.0</td>
<td>Time on suction (days)</td>
<td>32.0</td>
</tr>
<tr>
<td>Gas composition (% CH₄)</td>
<td>29.0</td>
<td>Borehole length</td>
<td>2.0</td>
</tr>
<tr>
<td>Reflectance Rv(max)</td>
<td>20.9</td>
<td>Suction pressure (kPa) median</td>
<td>1.8</td>
</tr>
<tr>
<td>Seam thickness</td>
<td>20.8</td>
<td><strong>Borehole orientation to stress</strong></td>
<td>0.0</td>
</tr>
<tr>
<td>Permeability (mD)</td>
<td>13.7</td>
<td><strong>Borehole orientation to cleat (100/280)</strong></td>
<td>0.0</td>
</tr>
<tr>
<td>Mineral component</td>
<td>10.6</td>
<td><strong>Borehole apparent dip</strong></td>
<td>0.0</td>
</tr>
<tr>
<td>Intertinite component</td>
<td>5.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture content</td>
<td>5.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ash content</td>
<td>5.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vitrinite component</td>
<td>5.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6 - Correlation results for data fitting borehole orientation and apparent dip criteria

As expected, there was no correlation between gas production and the three variables used to refine the dataset. The controllable variable, ‘time on suction’, maintained the strongest correlation with total gas production.

Given the relatively narrow range of apparent dip (0 to +1.6 degrees) the dataset was expanded to cover an increased apparent dip range, considered to provide greater flexibility for mine site deployment. The results of the statistical analysis on the dataset satisfying the criteria, listed below, are provided in Table 7.

- Borehole orientation to cleat (100/280) – 20 to 70 degrees
- Borehole orientation to stress – 0 to 50 degrees; and
- Borehole apparent dip – 0 to +3.25 degrees.
### Table 7 - Correlation results for data fitting borehole orientation and apparent dip criteria

<table>
<thead>
<tr>
<th>Factor assessed for correlation</th>
<th>R²</th>
<th>Factor assessed for correlation</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas composition (%CH₄)</td>
<td>33.0</td>
<td>Time on suction (days)</td>
<td>20.0</td>
</tr>
<tr>
<td>Gas content (m³/t)</td>
<td>30.0</td>
<td>Borehole length</td>
<td>6.7</td>
</tr>
<tr>
<td>Reflectance Rᵥ(max)</td>
<td>21.0</td>
<td>Borehole apparent dip</td>
<td>3.0</td>
</tr>
<tr>
<td>Seam thickness</td>
<td>21.0</td>
<td>Suction pressure (kPa) - median</td>
<td>2.0</td>
</tr>
<tr>
<td>Permeability (mD)</td>
<td>14.0</td>
<td>Borehole orientation to stress</td>
<td>1.3</td>
</tr>
<tr>
<td>Mineral component</td>
<td>11.0</td>
<td>Borehole orientation to cleat (100/280)</td>
<td>1.0</td>
</tr>
<tr>
<td>Intertinite component</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ash content</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture content</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vitrinite component</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note: R² represents the percentage variation of the variable relative to the regression model for the dataset. The higher the R², the better the regression model fits the data.*

The results show the controllable variable, ‘time on suction’, continues to maintain the strongest correlation with total gas production.

As shown in the results of the statistical analysis above, of all the non-controllable factors considered, both gas composition and gas content have the strongest correlation with total gas production. Figure 7 shows the distribution of total gas production relative to both gas composition and gas content.

**Figure 7 - Distribution of gas production data relative to gas composition and content**

Although both graphs show increasing total gas production with increasing composition and content, the gas composition appears to be the most dominant. In areas where CO₂ is the dominant gas, the total gas production is generally low, typically less than 200,000 m³. In the areas where CH₄ is the dominant gas, far greater total gas production is achieved, particularly where the CH₄ composition is greater than 80%. It must however be recognised that although it is possible to achieve very high gas production in the high CH₄ zones, there are also a significant number of very poor producing boreholes.

The dataset was refined on the basis of boreholes whose gas composition ranged between 0 and 40 % CH₄. Table 8 shows the results of the statistical analysis on this dataset. It can be seen that ‘time on suction’ continues to maintain the strongest correlation to total gas production of all the controllable factors considered. Suction pressure also has a far greater correlation to total gas production in the high CO₂ zones.

The results also show increased correlation between a number of the non-controllable factors and total gas production, in particular ‘permeability’ and ‘moisture content’.
Table 8 - Correlation results for data fitting the high CO2 criteria (0–40% CH4)

<table>
<thead>
<tr>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeability (mD)</td>
<td>77.0</td>
<td>Time on suction (days)</td>
<td>54.0</td>
</tr>
<tr>
<td>Gas composition (% CH4)</td>
<td>43.0</td>
<td>Suction pressure (kPa) - median</td>
<td>42.0</td>
</tr>
<tr>
<td>Moisture content</td>
<td>36.0</td>
<td>Borehole apparent dip</td>
<td>9.0</td>
</tr>
<tr>
<td>Mineral component</td>
<td>26.0</td>
<td>Borehole length</td>
<td>0.0</td>
</tr>
<tr>
<td>Gas content (m3/t)</td>
<td>22.0</td>
<td>Borehole orientation to stress</td>
<td>0.0</td>
</tr>
<tr>
<td>Vitrinite component</td>
<td>21.0</td>
<td>Borehole orientation to cleat (100/280)</td>
<td>0.0</td>
</tr>
<tr>
<td>Reflectance Rv(max)</td>
<td>20.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seam thickness</td>
<td>16.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intertinite component</td>
<td>13.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ash content</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: $R^2$ represents the percentage variation of the variable relative to the regression model for the dataset. The higher the $R^2$, the better the regression model fits the data.

A similar analysis was conducted on a refined dataset covering all borehole data within the gas composition range of 70 to 100% CH4. Table 9 shows the results of the statistical analysis on this dataset. Similar to the previous analysis, ‘time on suction’ continues to maintain the strongest correlation to total gas production of all the controllable variables considered. There is generally low correlation between all of the non-controllable factors and total gas production which is due to the high degree of variability in the total gas production of the boreholes in the high CH4 zones.

Table 9 - Correlation results for data fitting the high CH4 criteria (70–100% CH4)

<table>
<thead>
<tr>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
<th>Factor assessed for correlation</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas composition (% CH4)</td>
<td>8.0</td>
<td>Time on suction (days)</td>
<td>29.0</td>
</tr>
<tr>
<td>Reflectance Rv(max)</td>
<td>8.0</td>
<td>Borehole orientation to stress</td>
<td>14.0</td>
</tr>
<tr>
<td>Moisture content</td>
<td>6.3</td>
<td>Borehole orientation to cleat (100/280)</td>
<td>12.0</td>
</tr>
<tr>
<td>Gas content (m3/t)</td>
<td>4.2</td>
<td>Borehole apparent dip</td>
<td>2.0</td>
</tr>
<tr>
<td>Seam thickness</td>
<td>0.6</td>
<td>Suction pressure (kPa) - median</td>
<td>1.0</td>
</tr>
<tr>
<td>Permeability (mD)</td>
<td>0.0</td>
<td>Borehole length</td>
<td>0.0</td>
</tr>
<tr>
<td>Mineral component</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vitrinite component</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intertinite component</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ash content</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: $R^2$ represents the percentage variation of the variable relative to the regression model for the dataset. The higher the $R^2$, the better the regression model fits the data.

Capability assessment was then undertaken to determine statistically the impact on total gas production which may be expected through optimising the UIS gas drainage program, based on the results of the gas data analysis. The two areas where improvement and optimisation were considered possible are:
- Borehole orientation; and
- Gas drainage system maintenance.

As discussed previously, the gas production data indicates an optimum borehole trajectory exists in relation to cleat direction, stress direction and apparent dip. Statistical capability analysis determined that a 54.88% increase in average total gas production may be achieved by maintaining all UIS boreholes to within the following criteria:
- Borehole orientation to cleat (100/280) – 20 to 70 degrees
- Borehole orientation to stress – 0 to 50 degrees; and
- Borehole apparent dip – 0 to +1.6 degrees.

Where the range of the apparent dip is increased to cover the range 0 to 3.25 degrees the capability assessment concluded that a 26.34% increase in average total gas production may be achieved. Also, some 82 boreholes (26.8% of the total boreholes), achieved low total gas production, less than 100,000 m³.
However, the statistical analysis did not find any relationship between the factors considered and poor production. Although not analysed in this study, regular problems with UIS boreholes were reported which included; ‘borehole blocked’, ‘borehole full of water’ and ‘no suction’. It is believed that through focussed system management, which includes regular assessment of borehole flow performance that holes which perform below expectation may be quickly identified and appropriate corrective action may be taken to improve drainage.

Statistical capability assessment was used to assess the impact on average total gas production by addressing the boreholes producing less than 100,000 m$^3$. The impact was determined through eliminating the low producing holes from the dataset. The result was a 56.28 % increase in average total gas production. At an estimated cost of $20,000 for each installed UIS drainage borehole, significant financial benefit and gas drainage effectiveness (m$^3$/$) could be realised through implementing measures to avoid failed and poor producing holes. Table 10 lists the results of this statistical capability assessment.

Table 10 - Results of statistical capability assessment of UIS borehole gas production dataset

<table>
<thead>
<tr>
<th>Capability assessment - Total Gas Production (m$^3$)</th>
<th>No. of samples</th>
<th>Mean Total Gas Production (m3)</th>
<th>Average production increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ALL Data</td>
<td>188</td>
<td>179,183</td>
<td>Status Quo</td>
</tr>
<tr>
<td>Data fitting the Orientation &amp; Dip condition</td>
<td>53</td>
<td>277,524</td>
<td>154.88%</td>
</tr>
<tr>
<td>Data outside the Orientation &amp; Dip condition</td>
<td>135</td>
<td>140,573</td>
<td></td>
</tr>
<tr>
<td>Data fitting the Orientation condition, Dip range increase to (0°-3.25°)</td>
<td>106</td>
<td>226,385</td>
<td>126.34%</td>
</tr>
<tr>
<td>Data outside the Orientation condition, Dip range increase to (0°-3.25°)</td>
<td>106</td>
<td>118,163</td>
<td></td>
</tr>
<tr>
<td>Data greater than 100,000m$^3$ total gas production</td>
<td>106</td>
<td>280,026</td>
<td>156.28%</td>
</tr>
<tr>
<td>Data less than 100,000m$^3$ total gas production</td>
<td>82</td>
<td>48,822</td>
<td></td>
</tr>
<tr>
<td>Data fitting the Orientation &amp; Dip condition &amp; &gt;100Km$^3$ production</td>
<td>38</td>
<td>368,996</td>
<td>205.93%</td>
</tr>
<tr>
<td>Data fitting the Orientation condition, Dip range (0°-3.25°) and &gt;100Km$^3$ total production</td>
<td>72</td>
<td>308,426</td>
<td>172.13%</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

This study identified that a significant portion of the drilling effort yields little benefit to the overall gas drainage effort. Of the 306 UIS boreholes studied, 118 (38.5%) were found to be either blocked, flooded or had an effective drainage life of less than 30 days prior to being intersected by compliance drilling or roadway development. A further 82 (26.8%) achieved total gas production of less than 100,000 m$^3$.

Of the six controllable factors, listed below, which were included in the analysis, ‘time on suction’ consistently recorded the highest correlation to total gas production. The other factors in the order of correlation significance include:

- Suction pressure
- Borehole orientation relative to cleat
- Borehole orientation relative to stress
- Borehole apparent dip
- Borehole length

In high CO$_2$ zones, the gas production achieved was typically low, at less than 200,000 m$^3$.

In the high CH$_4$ zones, there was significant variation in the total gas production, ranging from extremely poor (~0 m$^3$) through to extremely high (~900,000 m$^3$). However, there was strong correlation between total gas production and ‘time on suction’ and borehole trajectory.

Borehole orientation and apparent dip have a significant influence on total gas drainage. Boreholes oriented 20 to 75 degrees relative to the dominant cleat, and 0 to 50 degrees relative to the major horizontal stress and with an apparent dip in the range of 0 to +1.6 degrees were found to have an average total gas production 54 % greater than the complete dataset and 97 % greater than those holes outside this range.

Also, by eliminating the poor producing holes (those achieving less than 100,000 m$^3$), implementing systems and measures to improve borehole integrity, and ongoing borehole and gas drainage system management, issues such as blockages, flooding and short lead times could be prevented. By improving the management and maintenance of the entire system, to reduce or eliminate the number of failed and poor producing boreholes, has the potential to increase the average total borehole gas production by up to 56 %.
Combining both optimised borehole trajectory and improved system management it may be possible to more than double the average borehole total gas production capability.

Although gas production in high CO₂ zones is impacted more by non-controllable factors than in the high CH₄ zones, however, finite improvement in gas production from UIS boreholes can be achieved by;

- Borehole maintenance, to avoid blockages, etc.;
- Optimise borehole trajectory – relative to cleat, stress and apparent dip;
- Maximise drainage time; and
- Maintain high suction pressures

ACKNOWLEDGEMENTS

The contributions of Matthew Jurak and Kate Lennox with the collection are recognised and greatly appreciated.

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ANALYSIS AND RESEARCH ON INFLUENCING FACTORS OF COAL RESERVOIR PERMEABILITY

Xiangchun Li\textsuperscript{1,2}, Baisheng Nie\textsuperscript{1} and Ting Ren\textsuperscript{2}

ABSTRACT: The permeability of coal is an important parameter in the theory of coal reservoir gas flow. Experiments have demonstrated that after the coal matrix adsorbs gas, adsorption swelling deformation will occur and when gas desorbs from coal matrix, it undergoes a shrinkage deformation. A new formula for estimating permeability is developed. By considering the adsorption swelling and desorption shrinkage deformation in this formula for permeability, the effect of coal matrix swelling and shrinkage on coal reservoir permeability is studied. The study shows that, as gas in the coal reservoir desorbs, gas pressure decreases and the coal matrix shrinks, so as to make swelling deformation smaller and increase the permeability of the coal reservoir. In addition, the influence of effective stress, Klinkenberg effects, burial depth and natural fracture on coal reservoir permeability is analyzed. The analysis and research is of significance to coal mine gas disaster prevention and coal bed methane gas resource development.

INTRODUCTION

The permeability of coal is an important parameter in the theory of coal reservoir gas flow. Many gas dynamic phenomena, such as coal and gas outburst, emission etc, are dependent coal reservoir permeability during exploitation. In this paper the effect of coal matrix swelling and shrinkage on coal reservoir permeability is studied by considering adsorption swelling and desorption shrinkage deformation in a new formula for estimating permeability. The influence of effective stress, Klinkenberg effects, burial depth and natural fracture on coal reservoir permeability is also analysed.

FACTORS INFLUENCING COAL RESERVOIR PERMEABILITY

Shrinkage effects of coal matrix

Many experiments have demonstrated that after the coal matrix adsorbs gas, adsorption swelling deformation will occur and when gas desorbs from the coal matrix, it undergoes a shrinkage deformation (Reucroft P J, Patel H, 1986; Seidle J P, Huitt L G, 1995; Levine J R, 1996). In the process of coal production the equilibrium of the original stress field and the original gas pressure field is disturbed, which lead to a stress redistribution near the mining space and results in gas flow. When the gas pressure falls below the critical desorption pressure, gas starts to desorp. With the decrease of pore pressure, gas desorption and micro-porous surface free energy increase, which results in the coal matrix shrinking and enhanced permeability of coal reservoir.

The increase of coal porosity will lead to the change of the permeability of coal and this affects gas seepage in the coal reservoir. According to the Kozeny-Carman equation, the change of permeability with porosity, $\varphi$, can be expressed as (Lu Ping et al., 2002)

$$ k = \frac{\varphi}{k_z S_p^2} = \frac{\varphi^3}{k_z S_v^2} $$

(1)

where $k_z$ is a dimensionless constant, $S_p$ is the pore surface area in a unit volume of porous media, $S_v$ is pore surface area in a unit pore volume of porous media.

In equation (1) the relation between porosity and adsorption swelling deformation can be expressed as (Li Xiangchun et al., 2005)

$$ \varphi = \frac{\varphi_0 + \varepsilon_p - \varepsilon_p}{1 + \varepsilon_p} $$

(2)
where \( \varphi_0 \) is original porosity in the coal reservoir, \( \varepsilon_V \) is the volumetric strain due to stress change, \( \varepsilon_P \) is the volumetric strain due to adsorption swelling, which can be expressed as (He Xueqiu et al., 1996)

\[
\varepsilon_P = \frac{ak \cdot RT}{V_0} \ln(1 + bp)
\]

where \( k_c \) is a proportion constant, \( V_0 \) is the volume of the mole of gas at STP, \( R \) is the gas constant, \( a, b \) are adsorption constants, \( T \) is the absolute temperature, \( P \) is the gas pressure in the coal reservoir.

Because adsorption of coal particles and molecular scale pore volume is not changed by the total stress, in the process of the change of the coal stress and strain it can be regarded that pore surface area of unit pore volume of porous media is constant (Lu Ping et al., 2002). It can be known from equation (1) that the change of coal permeability is mainly caused by the change of porosity. Substituting equation (2) into equation (1), the relation between permeability and adsorption swelling deformation can be expressed as

\[
k = k_0 \left( 1 + \frac{\varepsilon_V - \varepsilon_P}{\varphi_0} \right)^3
\]

where \( k_0 \) is original permeability in the coal reservoir.

It can be seen from equation (3) and equation (4) that as coal seam methane desorption pore pressure decreases and coal matrix shrinks, it will in turn lead to adsorption swelling deformation decrease and permeability increase. For the same coal sample and under the same conditions the more gas adsorbs, the greater the adsorption swelling deformation and the lower the coal permeability. The reason is that coal permeability is only related to the mesoporous, macroporous and fracture. When the swelling deformation of coal can not be generated along the outward direction under confined pressure the adsorption swelling deformation in micro-cracks and micro-pores will develop along the inward direction. Thus, the pore volume will decrease and gas flow is affected. In addition, adsorption gas will occupy parts of gas channel which, results in pore cross section decrease and coal permeability lower.

### 2.2 Klinkenberg effects

The important difference between the gas flow and the liquid flow in porous media is the phenomenon of gas slippage in solid walls. The essence of slippage is that because the gas molecular mean free path is equivalent to fluid field characteristic scale, the fluid molecules interact with the capillary wall which results in gas molecules slip along the porous surface and molecular velocity increase. For different materials the physical process is different, such as fluid surface adsorption, evaporation after gas condensates on the surface. In the seepage mechanics the interaction between the gas molecules and solid molecular is known as Klinkenberg effects (Klinkenberg L J, 1941). The permeability formula is (Joachim Gross, et al, 2003)

\[
k = k_s \left( 1 + \frac{b_s}{P_m} \right)
\]

where \( k_r \) is relative permeability, \( k_s \) is absolute permeability, \( P_m \) is gas mean pressure, \( b_s \) is a slippage factor, which can be expressed as (Joachim Gross, et al, 2003)

\[
b_s = 4c \frac{\lambda p_m}{r}
\]

where \( c \) is a proportion factor, \( r \) is the mean radius of a pore, \( \lambda \) is the gas molecular mean free path, which can be expressed as (Joachim Gross, et al, 2003)

\[
\lambda = \frac{k_b T}{\sqrt{2 \pi d^2}} P_m
\]

where \( k_b \) is Boltzmann’s gas constant, \( d \) is the molecular diameter, \( T \) is the absolute temperature.

It can be seen from equation (5) that the permeability increment caused by Klinkenberg effects is (Joachim Gross, et al, 2003)

\[
k_{slippage} = k_0 b_s / P_m
\]
Because of the existence of Klinkenberg effect, coal reservoir permeability will increase. The experiment research on aerosol permeability shows that the permeability increases with the decrease of porous fluid pressure (Joachim Gross, et al, 2003). The phenomenon of the experiment can be explained by Klinkenberg model. In general, coal seam methane flow in cleat can be described by Darcy seepage equation in which a stagnant gas layer near fracture wall is assumed. But under low pressure the gas layer will slip which result in higher actual gas production than the calculated value by equation. According to adsorption isotherm, the lower the gas pressures in coal reservoir the more coal seam methane desorption and production caused by per pressure decrease. Thus, with gas pressure in coal reservoir decreases Klinkenberg effect will have much influence on coal bed methane desorption and production.

2.3 Effective stress

The relation between the pore fluid pressure and rock skeleton deformation is based on the principle of effective stress, which was developed by Terzaghi. A simple effective stress formula can be expressed as

\[ \sigma_{ij} = \sigma_{ij}' + p \delta_{ij} \]  

where \( \sigma_{ij} \) is total stress, \( \sigma_{ij}' \) is the effective stress, \( p \) is the pore pressure, \( \delta_{ij} \) is the Kronecker Delta symbol.

A simple relation formula is reflected by the above effective stress formula among the pore pressure, the total stress and the effective stress. The important significance of the simple relation formula is that the study on the complex deformation of the porous media is equivalent to the study on no pore equivalent deformation under effective stress. The increase of effective stress can lead to fracture width decrease, pore closure and the permeability drop. Somerton (1975) established the relation between permeability \( k \) and effective stress \( \sigma_{ij}' \) by experiment research, which can be expressed as

\[ k = 1.03 \times 10^{-0.31\sigma_{ij}'} \]  

It can be known from equations (10) that there is a power function relation between the effective stress and permeability. The permeability will decrease with effective stress increase.

Another formula is given by Mckee et al. (1987).

\[ k = k_0 \cdot e^{-3C_p \Delta \sigma} \]  

where \( \Delta \sigma \) is effective stress increment, \( C_p \) is pore volume compressibility coefficient.

It can be seen from equation (9) that during gas exploitation effective stress increases as pore pressure decreases. It can be seen from equation (10) and equation (11) that with effective stress increase some cleats in the coal seam will close, which results in porosity and permeability decrease. The same conclusion was gained in experiment by Jiang Deyi et al.(1997). Figure1 shows the permeability decreases with effective stress increase.

![Figure1 - Relation between permeability and effective stress (Jiang Deyi et al., 1997)](image-url)
2.4 Burial depth

In general, the deeper the coal reservoir is buried the smaller coal reservoir permeability becomes. The change of coal reservoir permeability with depth is a function of stress. Only when different stress environment is differentiate, the change of coal reservoir permeability with depth is analyzed. In China it is very obvious that coal reservoir permeability decreases with stress increase. The researches show that coal reservoir permeability relates to antique and contemporary stress field, structural position, coal petrography and coal rank. But the total change trend of coal reservoir permeability is that the deeper coal reservoir is buried the lower coal reservoir permeability becomes.

Ye Jianping et al. (1999) finds that the change of coal reservoir permeability with depth in different area is different. In China there are three kinds of relation between coal reservoir permeability and burial depth. First, the deeper coal reservoir is buried the smaller coal reservoir permeability becomes, such as Hongyang, Huainan, Huaibei, Qingshui, the east of Taihang hill and Hancheng. Second, the deeper coal reservoir is buried the bigger coal reservoir permeability becomes, such as Shouyang area. Third, coal reservoir permeability is not changed with the depth.

2.5 Natural fracture

Coal is a rock with porous and natural fractures, which have important influence on gas adsorption, desorption and flow. Because natural fractures are the main flow channel in coal and coal reservoir permeability is controlled by natural fractures to a large extent, in direction of fractures growth the permeability of coal reservoir is several times higher than that in the vertical direction. In theory, the growth of natural fracture of coal reservoir benefits to improve coal reservoir permeability. From 1941 to now Hydraulics is studied by fracture made by all kinds of materials by Louis, Witherspoon, Tian Kaimin, Zhang Youtian, Tao Zhenyu, et al.. In the researches the cubic law in which fractures is regarded as parallel smooth plate model is used the most frequently. The cubic law can be expressed as (Zhen Shaohe, et al., 1999)

\[ q = f w^3 \]

where \( q \) is seepage velocity, \( w \) is fracture width, \( f \) is coefficient.

It can be seen from equation (12) that the bigger fracture width is the easier gas flows. The permeability of coal is measured under confined pressure in laboratory. The results show that coal permeability is extremely sensitive to stress and the permeability of coal decreases sharply with confined pressure increase. When the confined pressure increases 10 times, coal permeability decreases 2-3 orders of magnitude (Zhen Shaohe, et al., 1999).

Apart from the above factors, geo-electric field, geothermal field, coal moisture content will affect the permeability of the coal reservoir also, but their effect mechanism to coal reservoir permeability is not very clear.

CONCLUSIONS

1) A new formula to estimate coal permeability was developed by considering adsorption swelling. It can be seen from analysis of this formula that as coal seam methane desorption adsorption swelling deformation decreases and coal matrix shrinks, it will in turn result in permeability increase.

2) The influence of effective stress, Klinkenberg effects, burial depth and natural fracture on coal reservoir permeability is also analyzed. The analysis and research is of significance to the prevention of gas disaster and the development of coal bed methane resource.

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THE ROLE OF GAS MONITORING IN THE PREVENTION AND TREATMENT OF MINE FIRES

Darren Brady

ABSTRACT: Queensland’s underground coal industry, as a whole, has arguably the best gas monitoring systems in the world. Each mine utilises real time, tube bundle and onsite ultra fast gas chromatograph systems. Queensland’s mining legislation has specific requirements for mine gas monitoring but there is no requirement for all three techniques. Industry has however identified the need for all three and adopted this as a standard, resulting in over sixty thousand gas results collected each day. Automated monitoring systems are programmed to alarm for gas concentrations, gas ratios and explosibility. These alarms are then used to initiate predetermined actions to take control of the situation and prevent the compromise of safety to workers and the loss of resources. Dedicated software packages have been developed to assist in the interpretation of the large volume of results generated. The real time systems are used for real time warning, essential for incidents such as belt fires. Tube bundle systems suit long term trending used for identification of the onset of spontaneous combustion or for the determination of explosibility during the routine sealing of worked areas. Gas chromatograph analysis is used to provide a complete analysis and provides results for hydrogen and ethylene, key gases used in the assessment of spontaneous combustion. It is also crucial during significant spontaneous combustion events and coal fires to use gas chromatography to determine the explosibility status of the underground atmosphere otherwise the severity of the situation is likely to be under estimated. This paper outlines the need for all three techniques for assessing the underground status and outlines advantages and disadvantages of each.

INTRODUCTION

The underground mining industry in Queensland, Australia is internationally recognized as having the best mine gas monitoring systems. These systems are the result of nearly 20 years of ongoing development and follow recommendations from inquiries into explosions at underground coal mines in Queensland and subsequent changes to mining legislation.

Each underground coal mine in Queensland utilises real time (telemetric), tube bundle and onsite ultrafast gas chromatographs (GC) to meet their gas monitoring requirements. The industry has realised that all three techniques are necessary for effective monitoring of the mine’s atmosphere. Each technique has advantages and disadvantages which must be known to those using the systems and those using the results to interpret the status of the underground environment.

Monitoring on its own will never prevent a mine fire or put it out if it starts. What it does offer is a means of identifying a problem early and subsequently an opportunity to take appropriate controlling actions. The earlier a problem is identified the better the chance of successfully dealing with the problem. The best chance of getting an early warning is by continual monitoring.

The successful application of mine monitoring systems requires the setting of appropriate alarms that trigger effective remedial actions. The mine must also implement effective maintenance and calibration procedures to ensure reliable ongoing operation of the mine gas monitoring systems if they rely on them for this early warning or in fact use results to assess any control measures they might implement during an event.

MONITORING TECHNIQUES

Real Time Monitoring

Real time sensor systems (telemetric systems) are ideal for telling us what is happening now. The sensors must be located where the gas needs to be measured, and the measurement signal is sent to the surface. This means having multiple sensors underground, and that these sensors are exposed to the harsh underground environment which is not ideal for precise analytical measurements. This is not really a major problem as these systems are used to detect step changes, such as the onset of a fire, a sudden increase in a seam gas in the general body or reduction in oxygen. They offer real time warning and are the best system for identifying a sudden event such as a belt fire. The situation is reported when it happens. Generally, sensors included are methane, carbon monoxide, carbon dioxide, and oxygen.

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These types of sensors employed underground tend to have limited measuring ranges: carbon monoxide is often only capable of being measured up to 50 ppm, methane to 5 % and carbon dioxide to several percent. This range is fine while no problems exist, and indeed to alert the onset of a problem. But if a fire or other major incident involving generated gases occurs, these sensors may quickly reach full scale and be unable to return a true indication of the concentrations.

Due to the environment these sensors are in and their characteristics, they are not as useful for long term trending as the other techniques. Most of these sensors require the presence of oxygen to work and are therefore unsuitable for monitoring areas of low oxygen concentration such as sealed or non ventilated goaves.

As each individual sensor needs to be calibrated regularly (at least monthly) they are not suited to being located for long term monitoring in inaccessible areas such as the goaf.

Some of these sensors also suffer from cross sensitivities, as the reactions they rely on to give a response can be common to other gases found underground, such as carbon monoxide sensors being cross sensitive to hydrogen sulphide and hydrogen.

In the case of an explosion it is likely that the real time monitoring system will be rendered inoperable, requiring other techniques for the determination of the status of the underground environment.

Part 7 of Queensland’s Coal Mining Safety and Health Regulation (2001) outlines the requirement for mines to have a gas monitoring system providing continuous monitoring of methane, carbon monoxide, carbon dioxide and oxygen at stated locations. The requirement for continuous monitoring as opposed to continual, which was previously stated, infers the requirement for a real time system, making this the only technique referenced in the legislation.

**Tube Bundle**

Tube bundle systems draw gas samples from designated sampling locations underground to the surface through plastic tubes using vacuum pumps and analysed sequentially using infrared and paramagnetic techniques. Gases measured are carbon monoxide, carbon dioxide, methane and oxygen.

Because the analysers are on the surface, tube can be located in the goaf as once positioned there is no requirement to access the end sampling point (although sample filters and water traps located out bye will require maintenance).

Tube bundle systems are suited to long term trending (provided the system is suitably maintained). Very good analytical equipment is available and can be housed in dedicated air conditioned rooms on the surface with the samples dried and passed through particulate filters prior to entering the analyser.

Generally systems are set up to measure oxygen, carbon monoxide, carbon dioxide and methane. Given their ability to measure carbon monoxide down to 1 ppm, the long term stability of these analysers and the frequent sampling, this technique is best for long term trending of carbon monoxide, and carbon monoxide make, to identify a spontaneous combustion event. With respect to measuring range, it is normally only carbon monoxide that presents problems, with most systems capable of measuring to only 1000 ppm. Because methane and oxygen concentrations can be measured over all expected concentrations ranges, this technique is the best for automated monitoring of explosibility of an area so long as a fire or heating doesn’t exist. This technique is best for monitoring explosibility during a routine sealing operation and for the early onset of any spontaneous combustion event.

To get this improved stability and analytical capability, the immediate availability of the results is sacrificed. The samples need to be drawn to the surface prior to being analysed, meaning the data being generated can be from samples collected from over an hour before. There is only one bank of analysers, so only one sample is analysed at a time. Depending on the number of tubes in the system and the programmed sampling sequence, each point may only be sampled once every thirty to sixty minutes. Add this to the time taken to draw the sample from underground, which may be as long as an hour and it is obvious this technique is not suitable for the instantaneous detection of an incident such as a fire.

To minimise delay time in sample analysis, even when a sampling location isn’t been analysed it is being drawn to the surface by purging pumps that just vent the sampled gas to atmosphere. As multiple points are drawn through these purging pumps it is important to balance the flows of each of the tubes to ensure that each of the tubes is being purged and not just the tube with the least resistance.

Because the analysers in these systems rely on infrared absorbance and paramagnetic attraction the gas matrix is not important, making this technique suitable for the analysis of gases from oxygen depleted areas such as the goaf. What must be remembered is that the measurement of oxygen using paramagnetic analysers is flow rate dependent and the flow from each tube must be balanced to be the same, including any calibration gases used. Otherwise it is possible that two locations could in fact have the same oxygen concentration, but because of more
resistance in one of the tubes, the flow through the analyser is at a lower flow rate and as such results in a lower reading than a location with the same concentration but flowing through the instrument at a faster rate.

Too often the maintenance of the tubes is overlooked and the monthly leak testing identified in Australian Standard “AS2290.3 Electrical equipment for coal mines – Maintenance and overhaul Part 3: Maintenance of gas detecting and monitoring equipment.” is not performed or not done as stated by the standard. If maintenance follows the method outlined in the standard, it is not only possible to confirm that no leaks exist but also determine an approximate time taken for a sample to reach the surface. The knowledge of the draw times of each tube is critical to adequately assess what is happening and how long ago it actually happened in an emergency situation.

In the event of a mine explosion the tube bundle monitoring system may still appear to be functional but the location from which tubes are sampling may not be the same, due to damage to the tubes. A good tube bundle system will include monitoring of the vacuum pressure in each of the tubes, so following an explosion this data can be used to determine whether a tube has been compromised or not. It is also useful during routine operation for identifying increased restriction or sudden leakage in a tube, both of which can compromise the operation of the system.

If the tubes are damaged and not providing any valuable information, it may be possible to make use of boreholes and connect new tubes to locations of interest as the surface equipment will still be operational (flame arrestors are installed at each end of the sampling tubes).

Another major advantage of the tube bundle system is that it draws the sample to the surface and any locations requiring further analysis such as GC don’t require any additional trips underground to collect samples. This is particularly advantageous in emergency situations when personnel may have been withdrawn from the mine and re-entry is prohibited.

Gas Chromatography

Gas chromatography, with regard to gas analysis, involves the separation of all sample components followed by their measurement on relatively non-specific detectors. Specificity is obtained by virtue of the separation process rather than detection.

The use of a GC expands analytical capabilities to include gases crucial in the interpretation of spontaneous combustion events, particularly ethylene and hydrogen. The GC provides a complete analysis of the gases expected underground and is the only one of the three techniques capable of measuring hydrogen, nitrogen, ethylene and ethane. Determination of nitrogen is particularly important for determining oxygen deficiency in some spontaneous combustion indicating ratios.

Similar to the tube bundle, problems exist with bringing the samples to the GC. The significance of time delays in getting results is dependent on what the results are being used for. GC is not going to be suitable for detection of a belt fire because of the time delay between collection of the sample and analysis, but the delay is acceptable for confirmation of other results or for evidence and trending of spontaneous combustion indicators.

The GC is not the best analytical technique for low concentrations of carbon monoxide, therefore this technique is not the preferred method for determination of carbon monoxide makes nor low general body concentrations. However, during a significant spontaneous combustion event, fire or following an explosion, it is the only technique that will allow us to make an accurate determination of the explosibility of the underground environment.

Like the tube bundle system, the gas matrix of the sample does not affect GC analysis. So long as appropriate calibration gases are available, this technique is capable of measuring gases at any concentration above their detection limit. This eliminates the problems seen with the other techniques, particularly for carbon monoxide concentrations greater than 1000 ppm.

The ultrafast gas chromatographs in use in Queensland mines allow the analysis of most the components expected underground in approximately 2 minutes. Since their introduction the number of routine samples analysed has increased significantly allowing the mine to build comprehensive background knowledge of the normal background composition of particular areas underground. This increased sampling and analysis regime has also increased the chances of identifying and deviation from what is normal and allows the early intervention to deal with any problems identified.

This increased speed of analysis is invaluable during emergency situations, particularly when assessing the safety of the underground atmosphere for re-entry or during re-entry by mines rescue teams. In these cases what makes this assessment more effective is that GC is onsite and can be operated by mine personnel. There is no delay in determining the status underground while waiting for external providers to arrive or transporting samples away from site for laboratory analysis.
Comparison of Techniques

Comparison of results obtained at the same monitoring location using the different techniques are shown in Figures 1-7. It can be seen in Figure 1, that oxygen measurements collected over 1 day showed much less variation in measurements made using the tube bundle than for the real time sensor which regularly varied by more than 0.3% (absolute) between measurements. These variations make calculation of oxygen deficiencies used in ratios used for indicating/assessing spontaneous combustion, unreliable.

The methane results collected over 1 week in Figure 2 show good correlation in the magnitude of methane and reasonable agreement with trends between the two techniques. Although the absolute carbon monoxide concentrations measured over 1 day in Figure 3 were different, the trend over the 24 hours was the same for the two techniques. This difference may be as a result of a difference in calibration between the two techniques or as a result of sensor drift (most likely the real time).

Figure 1 - Real time vs tube bundle oxygen measurements

Figure 2 - Real time vs tube bundle methane measurements

Figure 3 - Real time vs tube bundle carbon monoxide measurements

Figure 4 - Tube bundle vs GC carbon dioxide measurements

Figure 5 - Tube bundle vs GC oxygen measurements

Figure 6 - Tube bundle vs GC methane measurements
Slight differences in absolute concentrations are evident and the difference in the amount of data collected by the techniques is significant, however the tube bundle data and GC data (Figures 4-7) show very good agreement over a one month period, reflecting increasing and decreasing trends. The difference in absolute concentration seen in Figure 7 could be a result of a difference in calibration gases used to set the instruments response.

![Tube Bundle vs GC carbon monoxide measurements](image)

**Figures 7 - Tube bundle vs GC carbon monoxide measurements**

When differences exist in the concentrations measured using the different techniques it can complicate the application of preset trigger levels. It also means that data for trends should only be generated by one technique and not an accumulation of results. When trends from different techniques are viewed they should indicate the same pattern. Often these measurement differences are related to the techniques themselves but some of the issues can be removed if calibration gases used for each technique are compared prior to being used to set instrument response.

Another strong point of real time sensors and tube bundle monitoring is that the monitoring points are at fixed locations, resulting in consistent automated sampling/measurement. When samples are collected underground for subsequent GC analysis, variations in results and trends can often be attributed to not collecting samples from exactly the same locations, or poor sampling techniques.

Gas chromatography is the only one of the techniques that actually measures nitrogen. As mentioned nitrogen is included in some ratios used to indicate spontaneous combustion. These ratios are still calculated from measurements made from the other techniques but when nitrogen values are required, the nitrogen concentration is assumed to be the balance remaining and calculated by summing the measured components and subtracting from 100. This presents obvious problems with the reliability of such calculations and in critical situations, calculation of these ratios should be done using GC results.

It is usual practice to confirm any abnormal results with one of the other techniques so prior to the need for any such comparison knowledge of how the mine’s own individual systems compare is required.

**ASSESSMENT OF FLAMMABILITY**

The need to perform a complete analysis by GC of atmospheres generated during coal fires or heatings is not only critical but the only option to obtain an accurate assessment of the flammability status of the underground environment.

Failure to do so can lead to wrongly assessing the atmosphere to be inert, when in fact it could be explosive or fuel rich, due to the generation of percent levels of carbon monoxide and hydrogen during mine fires. The presence of percent levels of these gases not only adds to the percentage of combustible gases present but also has a major influence in the lowering of the oxygen nose point (the lowest oxygen concentration at which an explosion can occur).

Figures 8 to 13 are examples of explosibility diagrams generated by GC analysis and tube bundle analysis of the same gas mix coming from heatings and mine fires. The composition of these samples is listed in Table 1. Because the tube bundle analysis does not include the hydrogen, and only includes up to 1000 ppm carbon monoxide, the percentage of combustible gas is underestimated and the calculation of the combustible (explosive) zone is incorrect. In a fire or heating situation, without the GC analysis an assessment of the flammability of the atmosphere underground is likely to be unreliable and can indicate that the atmosphere is inert when in reality it is explosive.

Figures 8 and 10 represent explosibility diagrams generated using results from tube bundle analysis during advanced spontaneous combustion events. If tube bundle results were used in isolation without consideration to the presence of hydrogen or concentrations greater than 1000 ppm of carbon monoxide then the atmosphere could
be assessed as being inert and rescue/fire fighting teams sent into the mine to deal with the situation. GC analysis of the same samples shows the atmosphere to be explosive (Figures 9 and 11) and considering the intensity of the spontaneous combustion an ignition source is likely to be present.

Figure 8 - Sample 1 as measured by tube bundle

Figure 9 - Sample 1 as measured by GC

Figure 10 - Sample 2 as measured by tube bundle

Figure 11 - Sample 2 as measured by GC

Figure 12 - Sample 3 as measured by tube bundle

Figure 13 - Sample 3 as measured by GC

<table>
<thead>
<tr>
<th>Gas</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
</tr>
</thead>
<tbody>
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<td>Hydrogen (%)</td>
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</tr>
<tr>
<td>Oxygen (%)</td>
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<tr>
<td>Carbon dioxide (%)</td>
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<td>Ethane (%)</td>
<td>0.0179</td>
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</table>

Table 1 - Mine fire samples

Figures 12 and 13 represent a sample collected from an advanced mine fire. In this case the explosibility diagram from the tube bundle data shows the atmosphere to be non combustible. This would mean that from this data only and with no consideration to the presence of hydrogen or carbon monoxide concentrations greater than 1000 ppm it would appear that it was safe to re-ventilate the area as there was not enough fuel for an explosion. The explosibility diagram generated by the GC data shows that the atmosphere is actually fuel rich, and the addition of
oxygen to the area could push the atmosphere through the explosive zone, again with a likely ignition source present.

These examples highlight the necessity to determine the explosibility of the underground atmosphere using gas chromatography.

**TRIGGER ACTION RESPONSE PLANS (TARPS)**

Mines determine gas levels that they think should not be exceeded. Often these numbers are based on historical data collected from the monitoring systems. To handle the large volumes of measurements made gas monitoring software has been developed that will automatically trigger a visual and audible alarm if one of the preset levels is exceeded. Alarm set points can be different for every sample point. These alarms activate what are known as Trigger Action Response Plans (TARPS) that have predetermined actions to follow. These actions have been formulated to ensure that appropriate actions are taken to ensure the safety of workers and maintain control of the mine.

Alarms can be generated from absolute concentrations, gas ratios or explosibility. They are most useful as an early warning, not as an alert to an emergency or a need to evacuate the mine. When alerted early enough the mine will have time to take remedial action to rectify the problem. Having continual automated gas monitoring provides the best chance for early detection.

The frequency and scope of monitoring is often included in the TARPS to ensure that the situation is not escalating or that the control measures are being effective. It must be noted that if inertisation is one of the control measures called for in the TARPS, any monitoring to determine the effectiveness of the control must be done from a location indicative of the affected area and not just at the point of entry of the inertisation gas, otherwise assessment of the situation may not be indicative of the true state.

**SUMMARY AND CONCLUSION**

As beneficial as they are, it must be remembered that monitoring systems on their own are not going to provide a successful solution to gas monitoring. Success depends on systems, processes and training built around the hardware and the way these systems are used.

- An effective gas monitoring system includes real time sensors, a tube bundle system and a gas chromatograph.
- Each technique is addressing different hazards.
- Each technique has strengths and weaknesses which must be known by those both operating them and using the generated results for interpretation.
- Effective maintenance and calibration procedures are required to ensure reliable ongoing operation of mine monitoring systems.
- Interpretation of data is best done looking at trends rather than one off samples. Even if the situation is being underestimated, any increase in intensity should result in an increase in the trend although the rate of change may not match the increase in intensity.
- Trends should only be based on data collected from the same technique.
- There are likely to be differences between in absolute measurements made using different techniques.
- A true indication of the flammability of the mine atmosphere during a mine fire, heating or post explosion can only be determined by gas chromatograph analysis due to the high concentrations of carbon monoxide and hydrogen possible.

**REFERENCES**


The Coal Mining Safety and Health Regulation 2001 Reprint No. 2C, 2007 the Office of the (Queensland Parliamentary Counsel).
PROBLEMS WITH DETERMINING OXYGEN DEFICIENCIES IN RATIOS USED FOR ASSESSING SPONTANEOUS COMBUSTION ACTIVITY

Darren Brady

ABSTRACT: Several common ratios used for determining spontaneous combustion activity rely on comparing the amount of particular products of oxidation with the amount of oxygen consumed to produce these products. Oxidation reactions become more efficient as the coal gets hotter, meaning more products produced for less oxygen consumed. There are many problems associated with accurately determining the true amount of oxygen used for use in these ratios. These problems relate to the monitoring technique (and its associated uncertainties) used to generate the results, deficiencies in the established equations utilised in calculations, dilution with other gases and other sources of oxygen depletion. Typically the oxygen deficiency is over estimated, resulting in the under estimation of the indicating ratio. For this reason the use of “one off” calculations to determine the status of the underground environment is not recommended. Instead trends of these ratios should be used to identify increases which indicate an increase in oxidation intensity. This paper outlines problems associated with the monitoring techniques and equations used to determine the oxygen deficiency.

INTRODUCTION

Oxygen deficiency is a term used for the amount of oxygen used (consumed/removed) from the inlet air stream by any activity as it undergoes reactions and interactions with the coal. Ratios utilising oxygen deficiency have been in use since at least 1921 (Cliff, Rowlands, Sleeman 1996) and are still valued tools in the identification and assessment of spontaneous combustion. In fact it is a requirement under Queensland’s mining legislation for mines to detect and calculate Graham’s ratio at certain locations. These ratios are used to measure the intensity of any oxidation of the coal that may be occurring. As the coal gets hotter the oxidation reaction becomes more efficient and more oxygen is converted to products of oxidation, such as carbon monoxide and carbon dioxide. Ratios such as Graham’s, Young’s and Jones-Trickett’s all divide products of combustion by the amount of oxygen consumed to give a quantifiable measure of how much oxygen was used to generate the amount of combustion products measured.

If there is more than one source of oxygen depletion than these ratios will be under estimated as it appears that more oxygen was used to produce the products than was really the case.

The measurement technique and the actual equation used for calculating the oxygen consumed by any oxidation/absorption-adsorption process also have a significant influence on the calculated values for these ratios. It is essential that anyone using these ratios to assess the status of the underground environment, understands the limitations and implications of both the analytical techniques and equations used to measure/determine the oxygen consumed. Graham’s ratio will be used to demonstrate these limitations and implications however it should be noted that the same are applicable to any ratio incorporating oxygen deficiency.

CALCULATING OXYGEN DEFICIENCY

Common Equations

Graham’s ratio is often expressed as

$$GR = \frac{100 \times CO_f}{0.265 \times N_{2f} - O_{2f}}$$

Where:

- $GR$ = Graham’s ratio
- $CO_f$ = final carbon monoxide concentration (%)
- $N_{2f}$ = final nitrogen concentration (%)
- $O_{2f}$ = final oxygen concentration (%)

---

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This format of the equation enables calculation of a result without actually knowing what the initial gas concentrations were. The denominator in Equation 1 represents the oxygen deficiency and is based on the assumption that nitrogen, being an inert gas, will not be consumed or created. If the initial gas entering the area under investigation had a fresh air ratio of 20.95 % oxygen to 79.02 % nitrogen (20.95/79.02 = 0.265), then equation 2 can be used to calculate the initial oxygen concentration by using the amount of nitrogen determined to be present in the sample;

\[ O_2 = 0.265 \times N_2 \]  

(2)

Where:

\[ O_2 \] = initial oxygen concentration (%)

\[ N_2 \] = final nitrogen concentration (%)

The measured oxygen concentration in the sample is then subtracted from the calculated initial oxygen to give the oxygen deficiency as in Equation 3.

\[ OD = 0.265 \times N_2 - O_2 \]  

(3)

Where:

\[ OD \] = oxygen deficiency (%)

\[ N_2 \] = final nitrogen concentration (%)

\[ O_2 \] = final oxygen concentration (%)

The use of the fresh air nitrogen concentration of 79.02% includes argon (Ar) in the amount and is used for techniques that are unable to differentiate the two gases. Use of this equation is only valid for samples where the initial gas has the same oxygen to nitrogen ratio as fresh air. Because we are using the measured nitrogen to determine the initial oxygen concentration, we eliminate most problems with dilution because the measured nitrogen will also been diluted.

Equation 3 can be used effectively only when the oxygen deficiency is not great. Because analysis is done on a percentage volume basis, if oxygen is being consumed/removed and nothing replaces it, although the actual number of nitrogen molecules does not increase, the percentage of the gas that is nitrogen does. This is often the case in a goaf where coal is left behind and oxygen is absorbed and adsorbed by the coal without producing oxides of carbon. This causes problems with the calculation of the oxygen deficiency, because of the elevated nitrogen concentration the calculated initial oxygen concentration is over estimated and therefore so is the oxygen deficiency. This is typical for samples collected from sealed or non ventilated areas. Examples from real mine samples highlighting this are outlined in Table 1.

Table 1 shows that when using Equation 2 in cases where significant oxygen deficiencies exist, the calculated initial oxygen can be greater than that in fresh air (20.95 %), an obvious problem. This over estimation of initial oxygen, results in an increased calculated oxygen deficiency which in turn leads to an under estimation of ratios using oxygen deficiency as the denominator. This is because it appears that more oxygen was used in the reaction and therefore was less efficient than is really the case.

Equation 1 is suited to situations where the initial gas concentrations are not available. With these ratios we are looking at how the gas concentrations have changed as they pass through an area. If initial gas results are available Graham’s ratio is often calculated using;

\[ GR = \frac{100(CO_f - CO_i)}{O_2 - O_2} \]  

(4)

Where:

\[ GR \] = Graham’s ratio

\[ CO_f \] = final carbon monoxide concentration (%)

\[ CO_i \] = initial carbon monoxide concentration (%)

\[ O_2 \] = initial oxygen concentration (%)

\[ O_2 \] = final oxygen concentration (%)

This equation is frequently used by mines that include a sampling point in an intake in its tube bundle gas monitoring system. Any problems associated with the calibration or drift of the oxygen analyzer are negated when
using this equation as they are common to both measurements. However if another monitoring point or sample (e.g. a longwall panel intake) is used as the initial oxygen, any dilution with seam gas is seen as oxygen deficiency and will over estimate oxygen deficiency and subsequently under estimate any indicating

Table 1 - Oxygen deficiency calculations

<table>
<thead>
<tr>
<th>O₂ (%)</th>
<th>N₂(+ Ar) (%)</th>
<th>Initial O₂ (%)</th>
<th>OD( %) Eq 3</th>
<th>Initial O₂ * (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.3</td>
<td>81.8</td>
<td>21.7</td>
<td>19.4</td>
<td>18.65</td>
</tr>
<tr>
<td>9.2</td>
<td>80.4</td>
<td>21.3</td>
<td>12.1</td>
<td>11.75</td>
</tr>
<tr>
<td>15.7</td>
<td>83.1</td>
<td>22.0</td>
<td>6.3</td>
<td>5.25</td>
</tr>
<tr>
<td>8.1</td>
<td>89.1</td>
<td>23.6</td>
<td>15.5</td>
<td>12.85</td>
</tr>
</tbody>
</table>

Table 2 - Graham’s ratio calculations using Equation 4

<table>
<thead>
<tr>
<th>CH₄ (%)</th>
<th>O₂i (%)</th>
<th>O₂f (%)</th>
<th>COf (%)</th>
<th>GR</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>20.95</td>
<td>20.8</td>
<td>0.0005</td>
<td>0.333</td>
</tr>
<tr>
<td>3%</td>
<td>20.95</td>
<td>20.8x0.97 = 20.18</td>
<td>0.0005x0.97 = 0.00049</td>
<td>0.063</td>
</tr>
<tr>
<td>6%</td>
<td>20.95</td>
<td>20.8x0.94 = 19.55</td>
<td>0.0005x0.94 = 0.00047</td>
<td>0.034</td>
</tr>
</tbody>
</table>

These results highlight how much ratios can be underestimated if oxygen deficiency is calculated using an actual oxygen measurement for initial oxygen, and the final oxygen concentration is affected by dilution. In this case dilution only, has led to a decrease in Graham’s ratio by a factor of ten. To overcome this problem the oxygen deficiency is best calculated as:

$$OD = \left( \frac{N_{2f}}{N_{2i}} \right) \times O_{2i} - O_{2f}$$

(5)

Where:

- OD = oxygen deficiency (%)
- $N_{2f}$ = final nitrogen concentration (%)
- $N_{2i}$ = initial nitrogen concentration (%)
- $O_{2i}$ = initial oxygen concentration (%)
- $O_{2f}$ = final oxygen concentration (%)

Multiplying the initial oxygen by the ratio of the final and initial nitrogen in Equation 5 takes into account any dilution and overcomes the problems demonstrated with Equation 4 as the nitrogen will have also been diluted. The same factor is applied to the initial carbon monoxide to give the equation:

$$GR = \frac{100 \times \left( \frac{COf - COi}{N_{2f}/N_{2i}} \right)}{\left( O_{2i} \times \frac{N_{2f}}{N_{2i}} \right) - O_{2f}}$$

(6)
Where:

\[ GR = \text{Graham's ratio} \]

\[ CO_f = \text{final carbon monoxide concentration} \%

\[ CO_i = \text{initial carbon monoxide concentration} \%

\[ N_2f = \text{final nitrogen concentration} \%

\[ N_2i = \text{initial nitrogen concentration} \%

\[ O_2f = \text{final oxygen concentration} \%

\[ O_2i = \text{initial oxygen concentration} \%

Table 3 shows how Equation 6 overcomes problems seen with Equation 4 caused by dilution when using initial oxygen concentrations. Initial concentrations of 20.95% oxygen, 79.02% nitrogen and 0 ppm carbon monoxide have been used.

Although using Equation 5 (and subsequently Equation 6) overcomes the problems with dilution seen in Table 2, it does reintroduce the problems seen in Table 1 where oxygen deficiencies are underestimated in cases where large oxygen deficiencies exist because of the increased measured nitrogen concentrations. The advantage in persisting with referencing other tube bundle monitoring points for initial gas values is that any drift or systematic errors with the analyzers are eliminated as they are common to both measurements.

Table 3 - Graham’s ratio calculations using Equation 6

<table>
<thead>
<tr>
<th>CH4 (%)</th>
<th>N2 (%)</th>
<th>O2 (%)</th>
<th>CO (%)</th>
<th>GR</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>78.8</td>
<td>20.8</td>
<td>0.0005</td>
<td>0.55</td>
</tr>
<tr>
<td>3</td>
<td>78.8x0.97 = 76.44</td>
<td>0.0005x0.97 = 0.00049</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>78.8x0.94 = 74.07</td>
<td>0.0005x0.94 = 0.00047</td>
<td>0.55</td>
<td></td>
</tr>
</tbody>
</table>

The Influence of Analytical Techniques

Neither tube bundle nor real time systems measure nitrogen. To calculate oxygen deficiency using any of the equations that require nitrogen input using these measurement techniques, nitrogen is calculated by summing the measured oxygen, carbon monoxide, carbon dioxide and methane, and assuming that all of the remaining gas is nitrogen.

There are problems associated with inferring the nitrogen concentration by difference when it is used for calculations using oxygen deficiency. Oxygen analyzers used in mining are required to meet tolerances of +/- 0.2 % (according to Australian Standard AS2290.3). Therefore, a measured oxygen concentration of 20.7 % could be as low as 20.5 % or as high as 20.9 %. If a sample returned concentrations of 10 ppm CO, 0.1 % CO2 and 20.7 % O2, the calculated nitrogen concentration would be 79.2 % (by difference). As the oxygen could be between 20.5 % and 20.9 %, the nitrogen (by difference) could be between 79.4 % and 79.0 %.

Calculating Graham’s Ratio using Equation 1

Graham’s Ratio could range between:

\[ GR = 100 \times 0.001 / (0.265 \times 79.4 - 20.5) \]

\[ = 0.1 / (21.04 - 20.5) \]

\[ = 0.1 / 0.54 \]

\[ = 0.18 \] and

\[ GR = 0.001 \times 100 / (0.265 \times 79.0 - 20.9) \]

\[ = 0.1 / (20.94 - 20.9) \]

\[ = 0.1 / 0.04 \]

\[ = 2.86 \]

If the interpretation was done in isolation on this sample, without allowing for the limits of accuracy of the analysers, the conclusion could vary between normal conditions and open fire (according to literature values for Graham’s ratio).
Oxygen measurements made using paramagnetic analysers tend to return much more stable results than those measured using electrochemical sensors employed in real time monitoring systems as can be seen in Figure 1.

Figure 1 compares measurements made at the same underground monitoring location using real time sensors and an oxygen paramagnetic analyzer employed in a tube bundle system. It highlights that oxygen measurements are much more stable using the tube bundle than for the real time sensor which regularly varied by more than 0.3 % (absolute) between measurements. Variations of the magnitude seen with the real time sensors have a significant impact on calculated oxygen deficiencies and subsequent ratios.

![Figure 1 - Real time vs tube bundle oxygen measurements](image1)

![Figure 2 - Real time vs tube bundle oxygen deficiencies](image2)

Figure 2 compares oxygen deficiencies calculated using Equation 3 for measurements made in a longwall return using real time and tube bundle data. The data shows that the tube bundle again returns the more stable data with the real time data frequently returning negative oxygen deficiencies, indicating the highly unlikely production, rather than consumption of oxygen.

Part 7 of The Coal Mining Safety and Health Regulation (2001) outlines the requirement for mines to have a gas monitoring system providing continuous monitoring of methane, carbon monoxide, carbon dioxide and oxygen at stated locations (including the return airway of each ventilation split). The gas monitoring system must also automatically detect and calculate the values of, amongst other things, the ratio of carbon monoxide to oxygen deficiency (Graham’s Ratio). The requirement for continuous monitoring as opposed to continual, which was previously stated, infers the requirement for a real time system. As seen in Figures 1 and 2, the variation in oxygen concentration using real time monitoring results in problems with calculating the oxygen deficiency that essentially make this monitoring technique unsuited to the calculation of Graham’s ratio in these locations.

Although the tube bundle appears much more stable it must be remembered that the measurement of oxygen using paramagnetic analysers is flow rate dependent and the flow from each tube must be balanced to be the same. Otherwise it is possible that two locations could in fact have the same oxygen concentration, but because of more resistance in one of the tubes, the flow through the analyser is at a lower flow rate and as such results in a lower reading than a location with the same concentration but flowing through the instrument at a faster rate. This variation will obviously affect the reliable determination of oxygen deficiencies and hence indicating ratios.

Gas chromatography (GC) is the only method of analysis that actually measures the nitrogen, giving added confidence in ratio calculations in light of the above. It must be noted that if oxygen deficiencies are being calculated using GC results, Equations 1, 2 and 3 must be modified if the results determine argon separate to nitrogen. All these equations assumed the oxygen to nitrogen ratio in fresh air to be 20.95:79.02 (0.265). In reality, 0.9% of that nitrogen total can be attributed to argon and if the two are being reported separately, the fresh air ratio is 20.95 % oxygen to 78.1 % nitrogen (20.95/78.1=0.268). Equation 1 should be modified to:

\[
GR = \frac{100 \times CO}{0.268 \times N_{2,f} - O_{2,f}}
\]

(7)

Equation 2 modified to:

\[
O_{2} = 0.268 \times N_{2,f}
\]

(8)

And similarly Equation 3 modified to:

\[
OD = 0.268 \times N_{2,f} - O_{2,f}
\]

(9)

If this is not taken into account, oxygen deficiencies and subsequent indicating ratios will be incorrect and in fact using the factor of 0.265 on samples that have been analysed with separate results for nitrogen and argon (GC), it
is possible in some cases to calculate initial oxygen concentrations less than the measured final oxygen concentrations — an obvious impossibility.

Minimum Oxygen Deficiency Required

Mitchell (1996) states that Graham’s ratio would be misused and could lead to incorrect interpretation when the oxygen deficiency is less than 0.3 %. Strang and MacKenzie-Wood (1985) also state, “This, like any calculation, is subject to limits of analytical errors and it is generally considered that oxygen deficiency of 0.2 percent or less would introduce gross errors. This point is made so that caution can be exercised in interpreting results when such low oxygen deficiencies occur.”

The calculated oxygen deficiencies for a typical longwall return in Figure 2 show the majority are less than the recommended minimum of 0.3% meaning use of ratios incorporating oxygen deficiency must be used with caution.

This problem associated with calculating Graham’s ratio when only small oxygen deficiencies exist also has implications for the requirement in Part 7 of Queensland’s “The Coal Mining Safety and Health Regulation (2001)” to automatically detect and calculate the value of carbon monoxide to oxygen deficiency (Graham’s ratio). When oxygen deficiencies are less than 0.3% (often the case in a longwall return) the variation and resolution of the measuring device contribute significantly to the calculated ratio.

Influence of Instrument Inaccuracies

When any analysis is performed there will always be slight inaccuracies in the measurements made, no matter how well the analysis was done and the how good the instrument performing the analysis is. These variations are totally acceptable and in fact expected. These slight variations from what may be the true concentration cause problems in samples with no significant oxygen deficiency whenever we get a slightly higher oxygen (or slightly lower nitrogen measurement by GC analysis), and apply the known fresh air ratio of oxygen to nitrogen to determine the oxygen deficiency.

When this happens there will be issues (mathematically) in determining Graham’s ratio (and any other ratios using oxygen deficiency). It appears that oxygen has actually been created, which goes against all that is known about the reactions of oxygen in the underground coal mining environment. What it really indicates is that the ratio has stayed the same. We have neither used nor created oxygen and that the difference comes totally from the acceptable inaccuracies (tolerance) of the measurement technique. Table 4 demonstrates this with results from GC analysis for samples collected in a longwall tailgate. Because analysis was by GC and nitrogen and argon reported separately Equation 9 is used to calculate oxygen deficiency.

Table 4 - Calculated oxygen deficiencies

<table>
<thead>
<tr>
<th>O₂ (%)</th>
<th>N₂ (%)</th>
<th>Oxygen Deficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20.61</td>
<td>76.63</td>
<td>-0.07</td>
</tr>
<tr>
<td>20.57</td>
<td>76.73</td>
<td>-0.01</td>
</tr>
<tr>
<td>20.33</td>
<td>75.84</td>
<td>0.00</td>
</tr>
<tr>
<td>20.23</td>
<td>75.71</td>
<td>0.06</td>
</tr>
</tbody>
</table>

Table 5 shows that some samples return a negative oxygen deficiency which will result in a negative value for any ratios using oxygen deficiency. It is often assumed that this means that there is a problem with the analysis, as it is impossible to create oxygen underground. A closer look at the results will explain what is happening.

In the first sample the nitrogen was measured to be 76.63 % and a negative oxygen deficiency obtained. If a result of 76.91 % had been returned the oxygen deficiency would have been positive. This represents a difference of 0.28 % (absolute) or 0.37 % (relative). This magnitude of variation is well within the tolerance expected for this analysis.

Similarly take the oxygen concentration measured as 20.61 % in the first sample, if in fact it had been measured as 20.53 % the oxygen deficiency would have been positive. This represents a difference of 0.08 % (absolute) or 0.39 % (relative). This is again, well within the tolerance expected for this analysis.
We can go even further and look at an even smaller decrease in the oxygen in combination with an increase in the nitrogen. For example if the oxygen came back as 20.57 % and the nitrogen 76.77 % (only very small changes) we would get a positive oxygen deficiency.

This is not peculiar to this sample but could be applied to all samples in the above table that returned negative excess nitrogen values. In general decreases in oxygen of less than 0.1 % or increases in nitrogen of approximately 0.2% will result in all samples with a negative oxygen deficiency becoming positive.

This just highlights that when we are analysing samples in which the oxygen to nitrogen ratio has remained constant, small, but expected analytical errors create problems ratio calculations that include oxygen deficiencies. This is why these ratios calculations are not suitable for these types of samples.

Regardless of how good the analytical equipment being used is, accuracy is influenced by the calibration gases used to set the response of the instrument. Calibration gas suppliers certify the composition of each component as the likely concentration with limits between which the true concentration lies. For example, the reported oxygen concentration in a recently supplied certified calibration gas is 19.6±0.5 %.

If used to set the response of the instrument to what we think is 19.6 % oxygen as indicated on the certificate, in reality it may be as low as 19.1 % or as high as 20.1%. This will result in all oxygen measurements being slightly high or low, but analytically acceptable. This is common to all of gases measured (according to their uncertainty).

A change in calibration gas can lead to a step change in values measured by the sensor/instrument calibrated with that gas. Procedures should be in place to check calibration gases prior to using them for calibration. The influence that this uncertainty has on the interpretation of results can be significant, particularly for ratios incorporating oxygen deficiencies. The oxygen could easily be reading 0.2 % high which will result in a much lower oxygen deficiency or could even result in what appears to be oxygen enrichment.

Instrument Variation

Figure 3 indicates the variation in the measurement of oxygen using the tube bundle. The points plotted are from two different tube bundle systems sampling fresh air on the surface. We can assume that the fresh air concentration of oxygen at these locations remains relatively stable and that the variations seen are as a result of the instrument measuring the concentration. The graph shows that the variations between the two systems are different, with one often returning changes of 0.1 % between samples. Although the other system did return some differences of this magnitude, most were of the order of 0.05 % or less. Variations of this magnitude can significantly influence any trending of ratios that incorporate an oxygen deficiency, as the calculated oxygen deficiencies can halve or double between readings. Figure 3 was for tube bundle data only, as no real time oxygen data from the surface was available. However, inference from Figure 1 Real time vs tube bundle oxygen data, would imply that the variation in oxygen measurements using real time sensors would make the problems with oxygen deficiency ratios worse. When the oxygen being measured is close to fresh air, even the resolution of the instrument can adversely affect these calculations.

SUMMARY AND CONCLUSION

Despite the problems highlighted in the text of this paper ratios incorporating oxygen deficiencies can still be very useful in the identification of the onset of spontaneous combustion or as an indicator of intensity during a spontaneous combustion event. Those involved in the interpretation must however be aware of all of these implications.

- Care must be taken when calculating oxygen deficiencies to ensure that the calculation is correct and representative for the sample and analysis technique.
- Ratios including Graham’s ratio that incorporate oxygen deficiency can be unreliable for samples where oxygen deficiencies are less than 0.3 %.
- Ratios incorporating oxygen deficiency will underestimate if there is more than one source of oxygen deficiency.
- Due to the variation in measurement and the small oxygen deficiencies present, real time monitoring is not suited to determining Graham’s ratio in longwall returns.
- Interpretation of data is best done looking at trends rather than one off samples. Even if the ratio is being underestimated, any increase in intensity should result in an increase in the trend although the rate of change may not match the increase in intensity.
- When any analysis is performed there will always be slight inaccuracies in the measurements made, no matter how well the analysis was done and the how good the instrument performing the analysis is. These variations are totally acceptable and in fact expected.
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The Coal Mining Safety and Health Regulation 2001 Reprint No. 2C, 2007 the Office of the (Queensland Parliamentary Counsel).


**TEN YEARS OF CONDUCTING LEVEL 1 SIMULATED EMERGENCY EXERCISES IN QUEENSLAND’S UNDERGROUND COAL MINES**

Martin Watkinson¹ Darren Brady¹

**ABSTRACT:** The inquiry into the explosion at the Moura No. 2 Mine in August 1994 recommended “Emergency procedures should be exercised at each mine on a systematic basis, the minimum requirement being on an annual basis for each mine.” (Windridge et al. 1996). In December 1996 the “Approved Standard for the Conduct of Emergency Procedures Exercises” was published and has been subsequently revised (Queensland Department of Mines and Energy Safety and Health Division 1999). This document provided guidelines for conducting mine site emergency exercises as well as the requirement for a test of state-wide emergency response by holding a Level 1 Mine Emergency Exercise at one mine on an annual basis. From 1998 to date (January 2008) ten Level 1 Mine Emergency Exercises have been held in Queensland. This paper discusses each of the exercises conducted to date as well as the logistics, scenario preparation, planning and organising required to conduct emergency exercises at this level.

**INTRODUCTION**

There have been ten Level 1 Emergency Exercises held in Queensland since 1998 and many more Level 2 and Level 3 exercises at all of the underground coal mines in Queensland as required by the “Approved Standard for the Conduct of Emergency Procedures Exercises”. The procedure followed for each Level 1 Exercises was as follows:

- Visit to the mine to determine local site conditions and hazards
- Determination of objectives of the exercise
- Development of alternative scenarios to test the objectives
- Ventilation modeling and preparation of gas data to reflect the scenario and objectives
- Full team meeting to brief all assessors and arrange for site inductions
- Conduct of exercise
- Debrief at mine site and report preparation
- Release of final report and dispersion of findings to industry.

The following sections briefly identify the scenario and some of the issues identified within the reports. Copies of all reports can be obtained in PDF format from simtars@dme.qld.gov.au.

**1998 SOUTHERN COLLIERY**

**Scenario**

The scenario was built around unauthorised alteration of a district regulator, affecting the ventilation flow on the operating longwall. The reduced flow, plus barometer fall resulted in a frictional ignition of methane. The resulting explosion destroyed and damaged several of the mines ventilation control devices. This exercise was conducted at 12:05 am to test call out of mines staff and responses in the early hours of the morning.

**Issues Identified**

- Communications between control room and Incident Management Team (IMT)
- Emergency call out procedures for staff off site
- Duty cards impossible to follow
- Need to conduct rapid evaluation of ventilation and gas trends
- Need for a clearly defined IMT process
- Need for adequately resourced IMT room
- Need for training in evacuation in zero/poor visibility
- Escape ways to be adequately maintained
- Safe havens for changeover of Self Contained Self Rescuers (SCSRs).

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1999 KENMARE COLLIERY

Scenario

This exercise was conducted at 3:43 pm and based on a conveyor belt fire in the intake that resulted in the whole of the mine being evacuated. The scenario was designed to test the IMT process and contractor training in emergency response. Unfortunately there were very few contractors underground at the time of the exercise due to the fact that they had been withdrawn prior to the commencement of the exercise.

Issues Identified

- Communications between control room and IMT
- Need for adequately resourced IMT room
- Need to conduct rapid evaluation of ventilation and gas trends
- Inflexibility in the duty card systems
- Need for training in evacuation in zero/poor visibility
- Training required for changeover of SCSR's
- Debriefing of evacuating personnel.

2000 NEWLANDS SOUTHERN UNDERGROUND

Scenario

This scenario was based on a transformer fire leading to the ignition of the coal ribs in the cut-through and was conducted at 21:00 on a Saturday night. The scenario was aimed to test the first response of personnel using Compressed Air Breathing Apparatus (CABA) as well as issues relating to call out of mine staff who traveled two hours away from the mine site to Mackay on weekends.

Issues Identified

- Call out of staff on a weekend when most are in Mackay
- Successful use of CABA as a first response tool
- Communications between control room and IMT
- Need for training in evacuation in zero/poor visibility.

2001 KESTREL MINE

Scenario

The Kestrel exercise was based on a massive roof fall on the maingate end of the longwall some 2.5 km inbye from the mains at 21:30 on a Tuesday night. The resulting air blast injured two of the longwall personnel. The remaining longwall personnel evacuated down the tailgate in an irrespirable atmosphere created by the lack of ventilation and normal seam gas make (carbon dioxide). This atmosphere was also above the 1.25% methane level for the operation of diesel vehicles.

Issues Identified

- Need for adequately resourced IMT room
- IMT process and data capture
- Need for a mines rescue vehicle for deployment into non-standard atmospheres
- Mines rescue deployment to recover injured personnel at distances of 2.5 km in an irrespirable atmosphere
- Issues of deployment of mines rescue personnel into hot and humid conditions
- IMT changeover process.

2002 NORTH GOONYELLA COAL MINE

Scenario

This scenario was run at 9:13 on a Monday morning with most of the mine staff present. It was designed to test the response with the mine manager and ventilation officer absent in Mackay (some two hours away by car) for “training”. The scenario was based on a vehicle fire which the operator could not put out. If a sufficient response was mounted the fire would have been deemed to be extinguished. There was also a simulated roof fall on the longwall, which allowed products from a spontaneous combustion deep in the goaf to report to the tailgate air
stream. Unfortunately the mine staff went into emergency ‘evacuation mode’ because of the ‘exercise mentality’ and forgot to tackle the fire. An adequate response was not mounted until six hours after the commencement of the exercise.

Issues Identified

- IMT process
- IMT changeover process
- Need for the staff to respond to the scenario and not just evacuate the mine
- Need for training in evacuation in zero/poor visibility
- Need for training in fire fighting procedures.

2003 CRINUM MINE

Scenario

The Crinum scenario was designed to raise awareness of the issues relating to the transport of gas cylinders, in particular acetylene. Acetylene cylinders can be very unstable if damaged. All mines should seriously review how they deal with acetylene and normal gas cylinders during transportation. The cylinder was “damaged” during a vehicle collision creating a vehicle fire, which subsequently developed into a coal fire. To completely pollute the intakes to the mine an air door between the intakes and the conveyor belt line was deemed to be left open for the purposes of the exercise. This again stresses the importance of conveyor belt segregation and the need for training/education to the workforce to ensure that all ventilation appliances are correctly used and any damage is reported. Without the simulation of this door being open, evacuation from the mine would have been much simpler along an uncontaminated conveyor road. An added complication for the mine was an underground visitor. “How readily do you think your mine would deal with an underground visitor in such a situation?”

Issues Identified

- Communications in and out of IMT
- Control room communications to personnel underground in the refuge bay (The men in the refuge bay sometimes got information before IMT)
- Recognition that burning coal can liberate explosive gases in sufficient quantities to create an explosive atmosphere in a mine with no methane seam gas
- The issues relating to inertisation with personnel underground
- The need for the ventilation officer/delegate to be free to do ventilation interpretation/modeling then feed the information back into IMT
- Safe transport of gas cylinders
- Need for well maintained and operated ventilation control devices.

2004 OAKY NO 1 MINE

Scenario

This exercise was based on a frictional ignition in a development panel resulting in a small coal dust explosion, “if such a thing exists”. All the men in the development panel were deemed to have been killed in the explosion. (These men also conducted a zero visibility evacuation to an area outbye of the explosion zone using training self rescuers while waiting for rescue teams to be deployed). The explosion was deemed to have destroyed several ventilation structures and polluted the other development sections of the mine. A series of photographs were prepared and each of the evacuating teams was briefed on what they would have seen as they traveled out of the mine.

One point of interest is that the exercise committee had made the decision that all personnel in the sandy creek area would have been killed by the initial “explosions”; however the last part of the exercise was where the mine located the “deceased”. This left the mine with a feeling of failure when in fact they had succeeded in taking the exercise to its fruition. This is an important part for anyone preparing industry scenarios and on reflection it would have been better if the mine staff could have at least found one survivor.

Issues Identified

- Trial of the Incident Control System/Mine Emergency Management System. (ICS/MEMS)
- The need for gas analysis and interpretation
- Debrief of personnel to get information into IMT
- Industry to recognize the need for adequate stone dusting to prevent coal dust ignitions.
2005 MORANBAH NORTH MINE

Scenario

The Moranbah North exercise was based on an Eimco traveling underground with a pod of Polyurethane (PUR) turning into a cut-through, hitting a transformer and causing a fire at 9:40 pm on a Sunday. The aim of the exercise was to highlight the requirements for fighting a PUR fire and the off gases produced, as well as to test the callout of mine staff on a Sunday evening. The location of the scenario enabled inseam response from an unaffected area.

Issues Identified

- The need for the mine to have a clearly defined and understood emergency response system
- The need for quick interpretation of the mine atmosphere and ventilation assessment
- Need for training in evacuation in zero/poor visibility
- Need for training and awareness when dealing with non-standard fires particularly PUR
- Automated call out procedures.

2006 BROADMEADOW MINE

Scenario

In order to create an evacuation scenario to test underground personnel a simulated fire in the intakes was coupled with a roof fall at the tailgate (T/G) end of the face preventing egress, forcing the men to evacuate inbye and undertake several changes of self contained self rescuers (SCSRs).

The exercise was based on a major fire on the hydraulic pump station and transformer in 4 cut-through in the maingate. The “fire” was caused by a catastrophic failure of the pumps/transformer and the burning oil and subsequent coal fire prevented egress out to the surface along the conveyor and travel road. The tailgate fall prevented egress through the longwall and along the tailgate roadway(s).

Issues Identified

- The needs for mines to develop a first response capability
- Improved training required in the donning and changeover of SCSRs
- Mines sites to evaluate the emergency response system they are going to use in light of the development of the Mine Emergency Management System (MEMS) by Queensland Mines Rescue Service (QMRS)
- Industry to adopt the draft recognized standard for the conduct of emergency exercises.

2007 GRASSTREE MINE

Scenario

Grasstree Mine has a high methane gas make on the longwall, is practicing methane drainage and utilises ventilation methods to keep the gas fringe away from the tailgate motor area of the working longwall. Nearby mines have all had frictional ignitions on both longwall and development panels. Consequently, it was decided to base the scenario for the 2007 Level 1 Mine Emergency Exercise on a frictional ignition on the longwall face. The exercise commenced at 20:00 on a Monday night to test the call out of site and other personnel.

For the purposes of the scenario it was deemed that the shearer drivers and another operator on the longwall would be seriously injured, however they would be able to make contact with the surface using non-verbal communication later in the exercise. The reason for this was to assist the decision process for the deployment of mines rescue teams.

Issues Identified

- All mines to conduct a gap analysis on their emergency response plans to recommendations made in 2007 report as well as previous exercises
- The draft recognized standard developed as part of the objectives of the 2006 Level 1 Mine Emergency Exercise should be circulated for comment and adoption as this will provide mechanisms for the follow up from Level 1 Mine Emergency Exercises and information not currently available from Level 2 mine site exercises
- Improved training required in the donning and changeover of SCSRs
- A mines inspector and industry safety and health representative should respond to and attend all Level 1 Mine Emergency Exercises.
DISCUSSION

The total number of assessors involved in ten years of emergency exercises is 180 giving an average of 18 assessors required per exercise. Assessors are invited from Queensland and New South Wales with representatives from Solid Energy New Zealand being present on the last four exercises. There is a core team of assessors on the management committee who have been involved in most of the exercises. One member has been present for all ten exercises. The management committee consists of representatives from:

- Simtars (Current Chair of the committee)
- Department of Mines and Energy Inspector
- Queensland Construction, Forestry, Mining and Energy Union Industry Safety and Health Representative (CFMEU ISHR)
- QMRS
- Minerals Industry Safety and Health Centre University of Queensland (MISHC)
- Mine site representative “mole”.
- With other assessors called in for the full team briefing. Organizations represented include:
  - Mine managers from Queensland coal mines, including the mine where next year’s exercise will be held
  - New South Wales (NSW) Mine Rescue Service
  - NSW CFMEU ISHR
  - NSW Mining Inspector
  - Solid Energy New Zealand
  - Compliance unit Department of Mines and Energy, Qld
- Around four “younger” mining engineers from Queensland mines are invited to join the assessment team each year to increase their awareness of emergency response requirements.

Approximately 200 man-days of professional time are involved in each level 1 process. That is from the initial site visit to the final printing and mailing of the report. This is a considerable investment in time and energy by coal industry professionals, but what has been gained by industry by undertaking these exercises and what are the issues that need to be addressed?

- Industry is sometimes slow to adopt or modify the emergency response plans after the running of the level 1 exercise. This can be seen by the number of times the same or similar recommendation has been made.
- All of the reports have identified issues with the training and donning of SCSR’s. An issue raised in the Sago Mine Disaster Report (J. Davit McAteer and associates July 2006).
- The amount of time taken to write and release the final report has varied from 1 month to over six months. The report often loses its impact if it is released several months after the exercise.
- The size of the reports (over 100 pages). Some personnel in the mining industry have indicated that the report is too large. Surely it is not too much to ask to read the executive summary, the conclusions and recommendations for a report which takes 200 man-days to compile?
- The mechanism of communicating the findings to industry in general. Different approaches have been tried from road shows, to articles in the Queensland Government Mining Journal to papers given at technical conferences such as this. None have proven to be the optimum solution.
- Some mines have approached the exercise as a pass or fail test generating a “Fear of the exercise”. The exercise committee needs to address this to ensure that all appreciate that this is a learning experience where everyone benefits. This is also covered in the draft recognized standard.
- The 2001 report on Kestrel Colliery recommended the development of a mines rescue vehicle. Subsequent research into an escape vehicle/rescue vehicle by Simtars funded by industry through the Australian Coal Association Research Program ACARP (AUS 800 K) has resulted in a design specification for a vehicle equipped with compressed air breathing apparatus (CABA), a sonic navigation system and onboard infrared methanometer. As yet this concept is still to be adopted, however several mining companies are prepared to provide further funding for the intrinsic safety (IS) certification of the navigation system. The other elements are now available commercially.
- Once a mine has undertaken the level 1 exercise it appears that the lessons learned and information are not freely available. For example one mine where the level 1 was held had 20 hard copies of the report mailed to them and not one copy could be found approximately 18 months later when a new owner took over the running of the mine.

We are living in an information age where possibly too much information is provided at once and electronically to be completely understood by the recipient.

Why are the safety management plans perceived to be the responsibility of the management team by the workforce? Ownership of safety management plans is required by the whole workforce, they need to realize this is
a way that they can impact on their own safety and in fact a mine with a good safety system often has good productivity as well. Being systematic is what improves safety and productivity.

**Logistics**

The logistics of coordinating hotel accommodation, transportation for up to 20 assessors is undertaken by a dedicated administration officer. Everything has to be arranged from food to take to site for the exercise, bus transportation, motel accommodation and booking of briefing rooms for the team. All of this has to be done with out alerting the mine site.

Note that for 20 people a 40 seat bus is required to take the additional work-wear, computers gas samples for testing the site gas analysis systems and associated food and clothing that is required.

Underground video footage has been taken by the site mole in the last two years. This simplifies the issues of site authority of gas detectors and taking non-intrinsically safe equipment underground. The voice-over for the footage is also best done by the site person who can best explain what is happening and what the current location is.

**Scenario Preparation**

The scenario preparation is undertaken by visiting the mine to assess site conditions and underground mining standards. The preliminary site visit takes place approximately five months before the anticipated exercise date.

Reference is made to previous exercises and high potential incidents in Queensland to determine the objectives of the current exercise, ie SCSR change over, first response call out of mines inspectorate and Simtars. Once these objectives have been agreed a scenario is proposed which will enable the objectives to be tested.

Ventilation modeling is undertaken in Ventsim, the Australian standard for coal mine ventilation modeling. This software contains a function where the percentage contamination from a single source event can be determined for the mine as well as the time taken for that pollutant to arrive at any working place/ monitoring point.

Data is then prepared for entry into Safesim (a Simtars program which acts as a dummy PLC) to feed real time gas information, for both real time and tube bundle analysers, into a duplicate Safegas system established on the mine site. Safegas is an industry standard for gas monitoring and interpretation in Queensland. The actual “severity” of the incident is sometimes tempered to ensure that mines rescue deployment can take place, ie ensuring that the gas levels are within the guidelines for the deployment of mines rescue teams in Queensland.

Once all ventilation modeling and gas pollutant levels have been established a meeting is called with three mine managers from other mines in the area. This is to validate the process and get industry “buy-in” to the objectives and selected scenario.

From there fine tuning is undertaken and information prepared for the full team briefing and site visit/inductions which are undertaken two weeks before the nominated exercise window. The venue for the 2008 Level 1 Exercise will be Newlands Northern Underground coal mine.

**Planning and coordination**

The planning of the exercises is now a streamlined process with ten exercises being completed. The amount of work is, however, underestimated by those who have not been involved in the process.

This is where dedicated administration support is required to compile contact lists, email information and coordinate motel room bookings etc to ensure that the base functions all run smoothly and prevent any adverse affects on the running of the exercise.

Each year the core team starts to prepare outline scenario’s for the following year along with the identification of the mine managers to be invited to the next exercise. As a rule the manger whose mine will be the subject of the next exercise is invited as an assessor in the year before his mine is the host mine.

Discussion is held between the core team as to the make up of each year’s assessment team. Around four younger mining engineers are invited to participate as part of the assessment team. There are no observers allowed, anyone present is given an activity to assess and provide input to the report.

A target two week window is identified for when the exercise is to be run. This is communicated to the mine site after the initial site visit and a mine site “mole” has been selected. The mine site mole is the point of contact for obtaining mine plans, ventilation models and copies of site safety management plans. These are all used to prepare briefing information for the underground assessors so that they have a pictorial view of the incident and the level of pollutants as they spread around the mine.
The briefing of the mine managers normally takes place one month before the full team briefing as this enables the scenario to be refined and briefing documentation to be prepared.

Two weeks prior to exercise window is set as the time for the full team briefing and site inductions. Assessors are normally given a visitor induction as they will always be in the company of mine site personnel during the exercise.

Once these dates have been set the chair of the exercise team starts to prepare a list of assessors and proposed deployments for the exercise to ensure that all elements of the mines emergency response can be evaluated. The team is requested to stay on after the exercise for a couple of days to complete their input to the report. History has proven that once personnel get back to mine sites information for the report is very difficult to obtain.

It then takes around 1 month, of dedicated time, to collate the individual assessor reports into one cohesive report which can be released to industry.

**SUMMARY AND CONCLUSION**

The running of emergency exercises for ten years in Queensland has led to many improvements in mine safety and emergency response. For example the introduction of change over bays at Grasstree mine in 2007.

A consistent theme is the requirement for training in emergency response, whilst there has been a dedicated thrust to adopt the MEMS/ICS process emphasis should be placed on the workforce to be able to don and change over a SCSR, to have confidence in the unit and be coached in decision making processes. This could alleviate the requirement to deploy mines rescue underground if everyone has successfully evacuated to the surface.

The Queensland Level 1 Exercise is a learning opportunity for the mines and state services to test their response, communication systems and interactions with the aim of continual improvement of the whole system.

With the large number of new employees in the industry due to the growth, some personnel do not appear to be fully aware their responsibility for their own safety and do not appear to have ownership of their escape systems.

Mine workers need to take ownership of the mine site hazard management plans in particular the mine emergency response plans. The need to be clearly aware of their requirement under the plan and ensure that they are completely competent in the donning and change-over process for SCSRs or other system that their mine has in place.

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DESIGNING FOR EFFICIENT INSTALLATION AND RELOCATION OF TRUNK AND PANEL CONVEYORS AT DONALDSON COAL, TASMAN MINE

Ross Middleton¹ and Mark Elliott²

Abstract: Mining operations at Donaldson Coal’s Tasman Mine were planned around maximizing the efficiencies in road way development and panel extraction within a mine to be established on traditional bord and pillar mining methods. Of particular relevance in early mine development, was that the installation of trunk conveyors stopped all coal production. Further, the mine plan required repeated relocation of production panel systems in relatively short cycles over the life of the mine.

A key element of the requirements identified by mine management was to minimize the non productive time that occurred during installation of trunk conveyors and the installation and relocation of panel conveyors. Aggressive targets were established for the time to be taken from hand over of a completed heading by the Mining team to Engineering team until hand back with an installed and operational conveyor. These targets, amongst others, necessitated a specific approach to the design of equipment with particular emphasis on simplicity of transport, minimized underground assembly and electrical termination, standardisation of parts and the ability to efficiently add to the installation following initial operation. A collaborative approach between mine engineering staff and the equipment supplier was established to share operational and design experience and develop practical design solutions to meet management’s targets.

This paper presents the criteria that were established for design constraints and required operational outcomes, the review of existing methods and the ultimate designs deployed at the mine. Several innovations were incorporated in the final designs to achieve the desired outcomes and these are presented.

Whilst the design criteria were specific to the Tasman operation, the results of the process and equipment features have likely applications in other mine operations seeking to improve development efficiencies and minimize conveyor installation times.

INTRODUCTION

Donaldson Coal’s Tasman Mine is located in the Lower Hunter area off George Booth Drive, West of the F3 Freeway and much of the lease lies beneath the Mount Sugarloaf Reserve. Mine life is expected to be 12 – 15 years with a total reserve of around 10 Million tones.

The target seam is the Fassifern seam. Due to the geometry and relatively small size of the deposit along with surface considerations, longwall mining methods were not considered economical or practical. Therefore the mine has been planned around bord and pillar mining methods with possible future production being supplemented by augering techniques.

Based on the bord and pillar mining method the mining layout comprises “main trunk” roadways developed throughout the lease and herringbone layout “panel” areas that side load onto the trunk system from numerous of areas. Roadways are nominally 3000 mm high x 5000 mm wide.

Economic production of this relatively small deposit at relatively low tonnages relies on cost effective infrastructure and efficient mining techniques. Further, the evolving mine design, based on issues of geology and production requirements meant that maximum flexibility was required in the configuration and re-use of the conveyor hardware.

Part of that planned efficiency has been identified in the safe transport, installation and commissioning times for the “trunk” conveyors and the similar recovery, relocation and installation of the “panel” conveyor units. The key parameter was the turn around time between access to a completed roadway and the hand over of an operating conveyor. Whilst rapid installation and commissioning was required on all conveyors, Trunk conveyors were expected to remain in position for life of mine where-as Panel conveyors would be recovered and relocated numerous times in the mine plan.

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This and other operating criteria for the equipment required a rethink on the traditional designs and existing equipment used in similar mine development applications.

**DESIGN CRITERIA**

The operational criteria for the conveyors was developed over a period of time by mine management and mine engineering staff along with input from the OEM supplier.

The anticipated mine layout, production levels and mining methods required conveyors using 1200 mm wide belt and either 75 kW or 150 kW installed power.

Belt speeds were assessed and predetermined at no more than 2.7 m/s. This conservative belt speed was adopted to minimize anticipated issues with low height transfers, general wear and tear on idlers and belt as well as providing reasonable scope to increase speed if needed due to unexpected or unspecified requirements in the mine.

The power and belt selections were supported by an assessment of likely life-of-mine requirements for trunk and panel conveyors in terms of lengths and lifts across the resource. Trunk systems would require up to 150 kW installed and panel systems only 75 kW to meet the expected conveyor geometries.

These ratings are typical of many “temporary development” systems used throughout the industry and are not in themselves significant other than to establish the base case for design. Of importance however is the realization that most of the equipment traditionally applied to such modest ratings was developed and used in the 1960’s or 70’s and would have typically involved the traditional style drives sold at the time by companies such as Huwood or Fox Manufacturing coupled with a separate loop take up and perhaps an integrated jib.

More recently, wheel mounted designs with loop take ups and drives have been developed with similar ratings and improved portability however they were considered unsuitable for the specific duty at Tasman. Tasman staff determined that design specific or “fit for purpose” equipment would be required to meet their application.

For example, whilst the Trunk conveyors would be left in position for the life of mine, they would be installed and extended in sequences and cycle times typical of development headings in other mines. Therefore the equipment design had to meet this dual functionality.

Beyond the normal criteria of design compliance, safety and operating parameters, the operational and other criteria established by the mine and relevant to this paper included:

- 48 hours Target time for installation, powering up and commissioning of a Trunk conveyor.
- 48 hours Target time for recovery, re-installation and commissioning of a Panel conveyor
- Typical set up for a new installation to be the starter, mechanical terminal set, structure, boot end and belting for a 100 metre initial length at hand over to the mining team.
- Panel conveyors to be able to load at any point on a Trunk conveyor.
- Trunk conveyors to be transom mounted trough idler design, optionally roof hung or floor mounted, panel conveyors to be suspended trough, floor mounted design but to have maximum commonality of components.
- Trunk structure design to provide simple “bolt free” construction to suit the development sequence.
- Minimised types of pulleys in the system.
- Maximum interchangeability and commonality of mechanical hardware in all conveyors.
- To suit future anticipated roadway limits the Panel conveyors had to be installed and able to transfer to a Trunk conveyor in no more than 2200 mm head height.

These criteria in combination required a “ground up” approach to the design of the conveyors and associated power and control systems.
DESIGN ASSESSMENT

Installation Time

In order to address the key criteria of limited installation or “hand over” construction times a review was undertaken of each element in a typical conveyor installation program to consider potential gains in the critical path.

The review necessarily involved OEM staff from the Mine Services division along with OEM design staff and mine operating personnel. The review identified major areas of concern or potential for improvement in traditional equipment designs or methods that could have a favourable impact on construction critical path.

The review identified the following:

- Preassembly
  - Piecemeal transport and assembly increases the “underground” time for installation therefore as much as possible should be pre-fitted before transport.
  - It is common for discharge assemblies to be installed separately and in particular chute work to be erected separately so an integrated, preassembled design has significant opportunity to reduce times.
  - Small parts add disproportionate times to installation. All incidental parts such as transition idlers, belt cleaners, brackets, electrical mountings should be pre-fitted.

- Electrical
  - Electrical fit up and testing can be a significant portion of the critical path.
  - Cabling and termination of electrical parts can not be easily accelerated underground due to limited access.

- Minimised Scope for Critical Period
  - In order to get away, the initial installation required only limited take up storage capacity. The design, if possible, should allow for the loop and winch to be installed in minimum length initially and extended at a later date without major interruption.

Panel Transfer

A similar review was undertaken on the requirement to be able to establish a transfer from panel conveyor to any section of the trunk conveyors and be able to relocate it efficiently with the panel move. The primary issue was the ability to construct an impact station at any (unknown) location on the trunk system. Traditional designs would not suit the objectives.

The issues identified in that review included:

- Minimised Scope
  - Breaking the belt or conveyor structure to affect the install is not attractive as it inevitably adds to the construction or recovery time.

- Portability
  - Accessibility to all points and both sides of the trunk system may be problematic so components should be ideally suited to allow safe man handling.

- Space
  - The specified clearance envelope for the panel transfer meant that the design must use minimum clearances

DESIGN RESULTS

The review process and subsequent detailed engineering phase led to several innovations in design for the various elements of the equipment. A summary of those features are:
Pre-assembly

The entire drive (Figure 1), jib module for the Trunk system is preassembled prior to underground installation. The chute work and belt cleaner sets are also pre-fitted. The jib assembly has a pinned and hinged design that allows the completed unit to fold into a low height, collapsed position for transport.

Figure 1 - The drive assembly incorporates fork tynes pockets in the base frame so that the unit can be easily carried into position underground before the unit is deployed.

Electrical

All electrical field devices are pre-fitted, cabled up and terminated to a common junction box. Prior to delivery or installation underground the entire assembly is wired and tested then broken down for transport. Wiring up underground requires only for the signal cable to be run and terminated to the J box.

Minimised Scope

If space or time limits require, the Trunk conveyor loop take up can be installed with a reduced number of modules. The winch is mounted at the inbye end of the assembly but on a carriage that can be detached and travel along the modules. Once further storage modules are required, they can be fitted to the inbye end of the loop and the winch slid back to the new inbye end. This feature allows the loop to be extended at any time without breaking the belt or re-roping the winch.

Panel System

The Panel conveyor system uses completely interchangeable mechanical and electrical components to the trunk system including drive module, pulleys, winch parts etc (Figure 2). The head pulley of the Panel is one of the loop pulleys of the Trunk system, where-as the head pulley on the Trunk is interchangeable with the larger drive pulley. There are only two pulleys designs used in entire system :

1. High tension bend / drive pulley (Live shaft)
2. Low tension bend / loop / tail / panel head pulley (dead shaft)

For compactness and portability the shaft mounted drive assembly of the trunk system including standard base plate, coupling and motor is mounted inboard in the Panel drive. A low speed chain drive is used to couple the gearbox to the drive pulley.

The Drive module incorporates a sliding, retractable jib frame and hinged impact plate / chute. The jib geometry is designed specifically to meet the very low head height specified in the design criteria.

The entire assembly is divided into separate, self contained modules that spigot together during installation.
Each section is mounted on wheels with integrated tow eyes at the inbye end of each section. Typically a panel installation will comprise a drive / discharge module and two loop modules. The winch incorporates the relocate able feature of the Trunk system so the loop can be extended at any time.

Figure 2 - The conveyor drive panel assembly

Panel Load Station

The load station developed so that a panel conveyor can load to any point on the trunk system is based on adding elements to the standard floor mounted conveyor structure to provide sufficient strength, the required impact idler spacing and the skirt and guarding requirements. The parts are all generally less than 25 kg each so can be managed by hand.

RESULTS IN THE FIELD

The results in practice have justified the effort put into specifying the stringent criteria and undertaking the reviews. Installation times have bettered the targets and operating performance has been excellent.

To date three Trunk systems have been installed and two Panel systems.

Key achievements to date include:

- The most recent Trunk 3 was installed and running in a total of three shifts including boot end and 120 metres of structure
- The most recent Panel 2 was installed and running in two shifts including boot, structure and belt.
- Idler life has exceeded expectations with zero bearing failures in the rollers since start of operations in September 2006. This result considered partly due to the modest belt speeds that were specified along with good seal design and accurate roller manufacturing tolerances.
Field experience has lead to some refinements in original designs:

- A “spoon” half has been introduced in the trunk to trunk transfers to improve material delivery and control.
- The impact half of the trunk transfer chute has been modified to incorporate additional curvature and improve flow.
- Ground clearance on the Panel conveyor assembly has been improved after the experience with Panel 1.
CONCLUSIONS

The specification and design process adopted for the Tasman Mine conveyors has highlighted the often underrated opportunity to improve mine efficiency by reductions in installation and relocation times of development conveyor systems.

The process relies on a “ground up” review of both installation process and equipment design in order to tailor the results to the stated objectives. No element can be considered a foregone conclusion as it is too easy to adopt the usual in the false expectation that longevity of practice is equivalent to ideal.

A collaborative approach to equipment design has significant benefits by allowing shared technical and operations knowledge to be applied to assessment and design evolution.
ABSTRACT: As the mining industry strives to become a zero defect/harm sector, the concept of risk management using Six Sigma quality management principles for consistency and standardisation of processes/actions and the effect thereof is currently being practiced in Indian coal mines. For monitoring of the effectiveness of actions as recommended under a risk management exercise, the process and corresponding defect are predefined in a statistical manner. A series of frequency distribution patterns and defects in statistical count are generated. The defects measured per million opportunities against each activity/process and thus the corresponding sigma level of process performance is applied. In order to build up system capabilities and graduate towards higher sigma levels of operation, the backbone exercise of Six Sigma management system is reached by carrying out the failure mode effect analysis (FMEA). Each potential failure mode component is assessed for its severity (S), occurrence (O) and detection (D). Detection is measured on an inverse scale of (1-10). To build up system capabilities in risk management, the recommendations of FMEA are implemented. Subsequently the potential failure mode component(s) are reassessed for their S, O and D. With every evolution in the system, as it slowly graduates towards becoming a Six Sigma risk management system, the risk priority number (RPN) should go on decreasing. A case study of a roof bolting exercise is presented as an example.

INTRODUCTION

Safety evolves gradually. Actions recommended under a risk management exercise, may have inherent variations in their effectiveness. Under Six Sigma, variations are measured in terms of standard deviation (sigma) and a six times of sigma (SD) is incorporated as a safety margin in the designed action plan. In Six Sigma parlance it is called “design for Six Sigma” (DFSS). This ensures that the action plan prepared under a Risk Management exercise is robust in its design by a six-sigma margin. Hence, any normal variations to the extent of its six times can be safely absorbed without any adverse affect on mitigation measures so adopted in the mine.

The Six Sigma quality management system standard is reached when only 3.4 defects/errors are tolerable out of one million performances of any activity. This corresponds to a correctness level of 99.99% or less than 3.4 defects per million opportunities (DPMO).

In risk management exercises, mining hazard identification and its risk ranking is done by a relevant/local mining team as a product of “consequences, probability and exposure” on a relative scale of (1-10). Six Sigma concepts put a halo to these steps of risk management by keeping a statistical surveillance on the effect of action the plan undertaken. The essence of Six Sigma is “what you measure that you get” and its success lies in precisely defining the process and the defect in physical and statistical form.

Six sigma implementation

Implementing Six Sigma involves several steps:
1. Defining the basic process (activity wise).
2. Define corresponding defect limit. Beyond safe zone would be called a “defect.”
3. Take repeated statistical observations of activities/measurements.
4. Observe pattern of data.
5. Plot its frequency distribution (normal distribution curve).
6. Measure its mean, deviations, and standard deviation (i.e. sigma).
7. Whether existing work practices accommodates six times of sigma or not?
8. Know statistically at what Sigma level, the current level of operation is.
9. Carry out FMEA and implement recommendations to reduce variability in process/ standard deviation.
10. Repeat steps 3 to 8.
11. Measure the reduced value of sigma (standard deviation) so that six times of S.D. is now accommodated with in safe margin, so designed, in steps I & II.

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12. With every graduation towards higher sigma level of operations, its process capability increases.

13. Figures 1 to 6 show the graphical representation of work process (1-6 sigma work processes).

Figure 1 shows the lower and upper specification limits of one sigma work process. One sigma means 690,000 defects per million opportunities. This is only 31% defect free output. The right hand side of the upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zones.

Figure 1 - lower and upper specification limits of one sigma work process

Figure 2 shows the lower and upper specification limits of the two sigma work process. Two sigma means 308,000 defects per million opportunities. This is 69.2% defect free output. The right hand side of upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zone.

Figure 2 - lower and upper specification limits of the two sigma work process

Figure 3 shows the lower and upper specification limits of the three sigma work process. Three sigma means 66,800 defects per million opportunities. This is 93.3% defect free output. The right hand side of upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zone.
Figure 3 - Lower and upper specification limits of the three sigma work process

Figure 4 shows the lower and upper specification limits of the four sigma work process. Four sigma means 6210 defects per million opportunities. This is 99.4% defect free output. The right hand side of upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zone.

Figure 4 - Lower and Upper specification limits of the four sigma work process

Figure 5 shows the lower and upper specification limits of the five sigma work process. Five sigma means 230 defects per million opportunities. This is 99.97 % defect free output. The right hand side of upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zone.
Figure 5 - Lower and upper specification limits of the five sigma work process.

Figure 6 shows the lower and upper specification limits of the Six Sigma work process. Six Sigma means 3.4 defects per million opportunities. This is 99.99% defect free output. The right hand side of upper specific limit (USL) and the left hand side of lower specific limit (LSL) are defect/rejection zones. The variations to Sigma process levels are shown in Figures 6a and 6b respectively.

Figure 6 - Lower and upper specification limits of the six sigma work process

Figure 6 a - The narrower the process width, the higher is the process capability and hence the higher sigma level of work process.
Figure 6a - Lesser the standard deviation of the process, more precise and consistent is the process.

Figure 6b - shows a comparison between a 3 sigma (93.3 %) and the 6 sigma (99.99966 %) work process. The 3 sigma curve is flatter while the 6 sigma curve is sharper/narrower showing better consistency in performance.

In a 3 sigma process the values are widely spread along the center line, showing the higher variation of the process. Whereas in a 6 sigma process, the values are closer to the center line showing less variation in the process.

Risk management process steps

Risk Management involves various steps as demonstrated in Figure 7. It gives a holistic representation of a risk management exercise, right from hazard identification to risk prioritisations, and leading to building up a detailed action plan and its implementation to monitoring its effectiveness. Six Sigma approach shall add halo by introducing measuring tools and techniques for statistical monitoring as well.
A risk management exercise was carried out on an underground mine of Coal India Ltd by the local mining team based on apprehension and its relative rating of “Consequences, Probability & Exposure” on a scale of one to ten (1-10) as shown in Table 1. Thus, the mine identified roof fall as the most important risk.

Table 1 - Risk management exercise of an underground mine of Coal India Ltd.

<table>
<thead>
<tr>
<th>No.</th>
<th>Description of Hazard</th>
<th>Consequences</th>
<th>Probability</th>
<th>Exposure</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Roof fall (Strata control)</td>
<td>4</td>
<td>10</td>
<td>9</td>
<td>360</td>
</tr>
<tr>
<td>2</td>
<td>Inundation due to incorrect mine plan</td>
<td>5</td>
<td>6</td>
<td>10</td>
<td>300</td>
</tr>
<tr>
<td>3</td>
<td>Mine Gases</td>
<td>3</td>
<td>5</td>
<td>10</td>
<td>150</td>
</tr>
<tr>
<td>4</td>
<td>Mine Fires</td>
<td>3</td>
<td>5</td>
<td>8</td>
<td>120</td>
</tr>
<tr>
<td>5</td>
<td>Explosives use</td>
<td>4</td>
<td>4</td>
<td>7</td>
<td>112</td>
</tr>
<tr>
<td>6</td>
<td>Mine Explosions</td>
<td>5</td>
<td>3</td>
<td>10</td>
<td>150</td>
</tr>
<tr>
<td>7</td>
<td>Transportation in mines</td>
<td>3</td>
<td>3</td>
<td>6</td>
<td>54</td>
</tr>
<tr>
<td>8</td>
<td>Electricity use</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>24</td>
</tr>
</tbody>
</table>

In the present paper, an attempt has been made to apply principles of Six Sigma risk management systems in a typical underground mine in Central Coalfields Limited (CCL), a subsidiary company under Coal India Limited. The process taken up for study is the roof bolting process – a common method of roof support used in any typical underground coal mine in India.

Description of roof support (by bolting) process

Preparation of support plan on the basis of method of work adopted, the physico-mechanical properties of strata, presence/absence of geological anomalies, and past work experience etc is carried out by mine manager and after getting it duly approved by the director (Mines Safety Inspectorate), it is circulated to assistant mine manager, supervisors, support personnel etc. with copies being posted at conspicuous relevant mine locations.

The mine is works on a conventional bord and pillar pattern with production of 300 t/d by drill and blast, and an average face advance of 1.2 m. After blasting, it takes around 30 minutes to get smoke cleared, depending upon ventilation efficiency of the mine. Then dresser dresses the neo-face, Mining Sirdar inspects the site for safety in terms of presence of gases, temporary stability of working etc, before he allows loading of coal from the blasted face. The guiding mantra is to never expose workers to unsupported working. Hence under temporary support arrangements, roof bolting preparations are made like drilling of holes as per support plan, insertion of roof bolt, cement capsules insertion, tightening with bearing plate/domed plate using torque wrench to get the desired strength of about 6 t/bolt. Its constant monitoring of strength is done by regular strength testing as well.
as destructive testing and records thereof are thoroughly maintained. Roof bolting must be done within 120 minutes of face exposure before the completion of initial roof adjustments that happens in roof strata just after blast. The following broad parameters are subsequently analysed after being measured statistically:

1. Hole depth;
2. Inclination to the bedding plane;
3. Spacing between holes/ bolts fitted there in;
4. Timing of bolt installation after face exposure;
5. Materials (roof bolts, bearing plate, cement capsules, etc) used.

Typical working parameters of the selected mine, include:

- Depth of working – 90 m;
- Incline access, 1 in 5 gradient;
- Method of work: bord and pillar, development; solid blasting.

Manual mining contemplating introduction of SDL/LHD with chain conveyor feeding to main trunk belt going up to surface should report:

- Degree of gassiness: Deg I, (make of gas < 1cum/t of coal output).
- Suspected old underground water logged bodies.
- Negligible geological anomalies.
- Production: 400-450 t/d.
- Age of incline: 5 years.
- Life of mine: 25 years.
- Development faces—5 (height of face 3 m, width 4 m), 1 in 22 level (east), 1 in 22 level (west), 1 in 21 Level (east), 1 in 20 level (west), 1 main dip.
- Rock mass rating RMR =58 (fair roof).

Roof support by roof bolting as per approved support plan with prescribed parameters as under:

- Roof bolt be installed in a grid of 1.2 m.
- Gap between first rows of bolts installed from side, < 0.8 m.
- Maximum Distance from last row of support and exposed face < 1.8 m.
- Hole depth 1.5m in middle of roadway, 1.2 m deep otherwise.
- Bolt of tor/mild steel; Diameter 22 mm, 1.5 m length, threading up to 125-150 mm, and using cement capsule 30-35 mm diameter of 500 mm length).
- Bearing plate of mild steel 6 mm thickness, and area 150 sq.mm.
- Nut Compatible with threaded bolt, hexagonal, at least 20 mm high.
- Timing of installation < 120-150 minutes from the neo-face exposure.
- Manual drilling; likely to introduce SDL mounted drill/ bolter, soon.
- Annular space between hole and bolt diameter be 8-12 mm.
- Support by support gang only, as per support plan.
- Anchorage strength of 3t / 5t (after ½ hr & 2 hr. respectively).

Defining defects with their corresponding weightings towards effectiveness of the roof bolting process are shown in Table 2.

<table>
<thead>
<tr>
<th>Defining Defects</th>
<th>Weightings assigned (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of hole &lt; 0.8m</td>
<td>20</td>
</tr>
<tr>
<td>Inclination of hole from normal to bedding plane &gt;10 degree</td>
<td>5</td>
</tr>
<tr>
<td>Spacing between holes &gt;1.5m</td>
<td>25</td>
</tr>
<tr>
<td>Delay Time of bolting from face exposed &gt;150 minutes</td>
<td>20</td>
</tr>
<tr>
<td>Quality of material &lt;6 in a scale of (1-10)</td>
<td>30</td>
</tr>
</tbody>
</table>

Monitoring of the aforesaid parameters is by Strata Management Cell only. Besides, workmanship of support personnel, testing and monitoring of bolt strength and determination of load build up on the installed bolts are critical for routine monitoring.
Methodology of study

A pilot study was carried out in the above mine in respect of 79 different roof bolts at its different working faces for two months each in two subsequent spells. The following parameters were measured. (see enclosed Excel sheet 1 of “databolting”):

1) Depth of hole (in meters).
2) Inclination of the access of the hole from normal to the bedding plane (in degrees).
3) Spacing between consecutive bolts (in meters).
4) Timing of bolt installation measured from exposure of the roof (in minutes).
5) Quality rating of the materials used(1-10 scale).

Measurement of defects

Definition of defects: The violations of the prescribed parameters were considered defects as statistically defined in Table 2.

Analysis of the Data

The sigma level of each of the activities during the field study was assessed and the results are shown in Table 3. Clearly, the sigma level of different processes was lower than the desired Six Sigma quality level. To improve upon the state of affairs, the backbone exercise of Six Sigma quality management principle and FMEA was carried out to identify the root causes of variability in the process. The impacts are measured in terms of "severity, occurrence and detection." on a relative scale of (1-10) Detection is measured on an inverse scale (Tables 5.1 -5.5).

<table>
<thead>
<tr>
<th>Sl.No.</th>
<th>Description of parameters</th>
<th>DPMO</th>
<th>DPMO</th>
<th>Corresponding Sigma level of operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Hole depth</td>
<td>(4/79)*10^6</td>
<td>50632</td>
<td>3.1</td>
</tr>
<tr>
<td>2.</td>
<td>Hole Inclination</td>
<td>(5/79)*10^6</td>
<td>63291</td>
<td>3.0</td>
</tr>
<tr>
<td>3.</td>
<td>Spacing between holes</td>
<td>(8/79)*10^6</td>
<td>101265</td>
<td>2.7</td>
</tr>
<tr>
<td>4.</td>
<td>Timing of bolt installation</td>
<td>(6/79)*10^6</td>
<td>75949</td>
<td>2.9</td>
</tr>
<tr>
<td>5.</td>
<td>Material quality</td>
<td>(8/79)*10^6</td>
<td>101285</td>
<td>2.7</td>
</tr>
<tr>
<td>6.</td>
<td>Overall process of roof bolting</td>
<td>(31/79*5)*10^6</td>
<td>78481</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Remedial measures taken

Remedial measures suggested under FMEA were implemented in mines and the entire roof bolting processes were re-run for similar 79 bolts carried out in subsequent two months of observation. Results are shown in Table 4 below.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Parameters</th>
<th>DPMO</th>
<th>DPMO</th>
<th>Corresponding Sigma level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Overall process of roof bolting</td>
<td>(1/79*5)*10^6</td>
<td>2531</td>
<td>4.5</td>
</tr>
</tbody>
</table>

CONCLUSION

By applying Six Sigma approaches, the process efficiency of roof bolting was improved from a level of 2.9 sigma to 4.5 sigma. Thus, the number of defects were reduced to 1 in 79x5 = 395 opportunities i.e. 2531 DPMO. Process capability increased and process width became narrower. With the same lower and upper specification limits, defect possibility is now much reduced. Thus the roof bolting processes were tending towards the Six Sigma quality level of operation.
Such an approach can be applied to other mining activities that have a bearing on safety. Extending further its domain, it can be applied to address occupational health dimensions as well, into its foray. Defining activities, processes and corresponding defects with its constant statistical surveillance can lead to achieving a status of zero harm industry to the mining sector.

REFERENCES

Design for Six Sigma – Mr. Greg Brue, Tata McGraw-Hill Edition
### Table 5 - Failure Mode Effect Analysis (FMEA)

#### Table 5.1 – Component - hole depth

<table>
<thead>
<tr>
<th>Function</th>
<th>Potential failure mode</th>
<th>Severity</th>
<th>Potential causes</th>
<th>Occurrence</th>
<th>Current Prevention</th>
<th>Detection</th>
<th>RPN</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>hole depth</td>
<td>Support men / drillers unaware of the specifications of hole depth</td>
<td>7</td>
<td>No training given</td>
<td>5</td>
<td>VTC in operation</td>
<td>6</td>
<td>210</td>
<td>Training to be provided as per new DGMS module 1999</td>
</tr>
<tr>
<td></td>
<td>Proper machine not provided</td>
<td>8</td>
<td>Lack of priority</td>
<td>6</td>
<td>Inventory management</td>
<td>7</td>
<td>336</td>
<td>Spare part management/ SCM to be implemented</td>
</tr>
<tr>
<td></td>
<td>Inadequate personnel</td>
<td>7</td>
<td>lack of supervision</td>
<td>7</td>
<td>Monthly Manpower planning</td>
<td>7</td>
<td>343</td>
<td>Weekly MPP, manager to monitor critical events himself</td>
</tr>
<tr>
<td></td>
<td>Incompetent drillers</td>
<td>8</td>
<td>Poor selection</td>
<td>8</td>
<td>Separate drillers category</td>
<td>9</td>
<td>576</td>
<td>Right selection</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Poor training</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Continuous training</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No diverting of people from critical to menial job</td>
</tr>
</tbody>
</table>
Table 5.2 - Component - hole inclination

<table>
<thead>
<tr>
<th>Function</th>
<th>Potential failure mode</th>
<th>Severity</th>
<th>Potential causes</th>
<th>Occurrence</th>
<th>Current Prevention</th>
<th>Detect</th>
<th>RPN</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>hole inclination</td>
<td>Support gang unaware of the importance of hole inclination</td>
<td>7</td>
<td>Poor training given</td>
<td>7</td>
<td>VTC in operation Inspection by Manager/OM</td>
<td>8</td>
<td>392</td>
<td>Training to be provided as per new DGMS module 1999</td>
</tr>
<tr>
<td></td>
<td>Proper machine not provided</td>
<td>8</td>
<td>Lack of priority Poor stores management</td>
<td>6</td>
<td>Inventory management</td>
<td>7</td>
<td>336</td>
<td>Spare part management/ SCM to be implemented</td>
</tr>
<tr>
<td></td>
<td>Goggles not provided</td>
<td>7</td>
<td>Priority less, poor health/hygiene concern</td>
<td>8</td>
<td>Stores arrangement Local purchase Drill bits /other support items</td>
<td>8</td>
<td>448</td>
<td>Implement ISO18000, Regular IME/PME Keep critical items in sufficient stock</td>
</tr>
<tr>
<td></td>
<td>Hand drilling</td>
<td>7</td>
<td>Manual mining</td>
<td>9</td>
<td>Primitive vision, low economic scale</td>
<td>9</td>
<td>567</td>
<td>Mechanisation must, Introduce earliest possible SDL mounted drilling machine/ Roof bolter</td>
</tr>
<tr>
<td></td>
<td>Inadequate personnel</td>
<td>7</td>
<td>lack of supervision</td>
<td>7</td>
<td>Monthly Manpower planning</td>
<td>7</td>
<td>343</td>
<td>Weekly MPP, manager to monitor critical events himself</td>
</tr>
</tbody>
</table>
### Table 5.3 – Component - spacing between holes

<table>
<thead>
<tr>
<th>Function</th>
<th>Potential failure mode</th>
<th>Severity</th>
<th>Potential causes</th>
<th>Occurrence</th>
<th>Current Prevention</th>
<th>Detection</th>
<th>RPN</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>spacing between holes</td>
<td>Support gang unaware of support plan/grid pattern</td>
<td>9</td>
<td>poor training given, poor workmanship</td>
<td>6</td>
<td>VTC in operation, Mining sirdar/OMmonitors</td>
<td>6</td>
<td>324</td>
<td>Training to be provided as per new DGMS module 1999,CMR 108,109, support plan</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Proper gadgets not provided</td>
<td>8</td>
<td>Lack of priority, Poor stores management, Poor work culture</td>
<td>6</td>
<td>Inventory management, Exclusive strata management cell operating in area</td>
<td>8</td>
<td>384</td>
<td>Strata management cell at mine level under asst. manager</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inadequate/ incompetent personnel</td>
<td>8</td>
<td>lack of supervision, unable to appreciate importance of job</td>
<td>7</td>
<td>Monthly Manpower planning, Poor work culture, Safety at back seat</td>
<td>7</td>
<td>392</td>
<td>Weekly MPP, manager to monitor critical events himself</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.4 - Component - time of bolt installation

<table>
<thead>
<tr>
<th>Function</th>
<th>Potential failure mode</th>
<th>Severity</th>
<th>Potential causes</th>
<th>Occurrence</th>
<th>Current Prevention</th>
<th>Detect</th>
<th>RPN</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timing of installation</td>
<td>Unaware of its importance of appropriate timing of support</td>
<td>9</td>
<td>No training given highlighting its importance</td>
<td>7</td>
<td>VTC in operation</td>
<td>8</td>
<td>584</td>
<td>Training to be provided as per new DGMS module 1999, support plan, vocational films to be shown</td>
</tr>
<tr>
<td></td>
<td>Late clearance of fumes after blasting</td>
<td>8</td>
<td>Poor ventilation</td>
<td>6</td>
<td></td>
<td>7</td>
<td></td>
<td>Afresh vent. survey</td>
</tr>
<tr>
<td></td>
<td>Dresser is not available</td>
<td>7</td>
<td>Poor work culture/discipline</td>
<td>6</td>
<td>Attendance system, separate category of dresser</td>
<td>5</td>
<td>210</td>
<td>Discipline, proper manpower distribution</td>
</tr>
<tr>
<td></td>
<td>Proper gadgets not provided to dresser</td>
<td>7</td>
<td>Lack of priority Poor stores management</td>
<td>7</td>
<td>Inventory management</td>
<td>5</td>
<td>245</td>
<td>Stores management, Spare part management/ SCM to be implemented</td>
</tr>
<tr>
<td></td>
<td>Late inspection by sirdar</td>
<td>8</td>
<td>Poor work culture</td>
<td>6</td>
<td>Asst mgr/Overman inspection Mine management</td>
<td>8</td>
<td>384</td>
<td>Safety committee, workmen inspector/inspections increase, meaningful</td>
</tr>
<tr>
<td></td>
<td>Support gang not present</td>
<td>8</td>
<td>Poor work culture/discipline</td>
<td>5</td>
<td>Manpower distribution, workmen inspector, safety week/drive</td>
<td>7</td>
<td>280</td>
<td>Dissemination on support plan, Training to supervisors, no diversion of persons</td>
</tr>
<tr>
<td></td>
<td>Support materials not available at site</td>
<td>9</td>
<td>Poor work culture, house keeping, indenting</td>
<td>7</td>
<td>No safety culture/priority</td>
<td>7</td>
<td>441</td>
<td>Training to supervisors about its importance</td>
</tr>
<tr>
<td></td>
<td>persons diverted from support gang to other jobs</td>
<td>7</td>
<td>Safety culture missing</td>
<td>6</td>
<td>No safety culture/priority</td>
<td>7</td>
<td>294</td>
<td>Training to supervisors about its importance</td>
</tr>
<tr>
<td></td>
<td>Geological anomalies encountered</td>
<td>8</td>
<td>Geo-informatics poor</td>
<td>3</td>
<td>Geologist operates from H.Q/Area</td>
<td>7</td>
<td>168</td>
<td>Geologist be in Strata mgmt cell of mine</td>
</tr>
<tr>
<td></td>
<td>Inadequate personnel in competence as well as number</td>
<td>8</td>
<td>lack of supervision</td>
<td>5</td>
<td>Monthly Manpower planning</td>
<td>7</td>
<td>280</td>
<td>Weekly MPP, manager to monitor critical events himself, constantly</td>
</tr>
</tbody>
</table>
### Table 5.5 – Component - Quality of materials used

<table>
<thead>
<tr>
<th>Function</th>
<th>Potential failure mode</th>
<th>Severity</th>
<th>Potential causes</th>
<th>Occur</th>
<th>Current Prevention</th>
<th>Detec</th>
<th>RPN</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old cement capsules with poor quality ingredients and hard polythene cover provided</td>
<td>Lack of supervision/ appreciation</td>
<td>9</td>
<td>Poor stores management / house keeping</td>
<td>6</td>
<td>Mine management/ strata management cell at Area</td>
<td>8</td>
<td>432</td>
<td>Surveillance of critical inputs materials under asst. Manager/ graduate civil engr. Paper wrapped capsules, use resin capsules</td>
</tr>
<tr>
<td>Steel bolts of inadequate/mismatch specifications and being unthreaded</td>
<td>Poor workshop/ supply source</td>
<td>8</td>
<td>Poor workmanship at workshop</td>
<td>6</td>
<td>Workshop at area level operating Under Dy Chief (Mech) Engg</td>
<td>7</td>
<td>336</td>
<td>Proper raw material input, shift monitoring by graduate mech/ming engr Trg. to foreman, operators ISO 9001 certification to w/shop Six sigma quality level at mfd. unit</td>
</tr>
<tr>
<td>Bearing plate not provided</td>
<td>Lack of its importance</td>
<td>9</td>
<td>Poor workshop/ supply source Lack of supervision</td>
<td>6</td>
<td>Monitoring by strata management cell and at mine by O/M, M/sirdar/ asst mgr/ mgr</td>
<td>7</td>
<td>378</td>
<td>Engineering culture and approach be inculcated Strata mgmt cell to be pro active in dissemination and hence implementation</td>
</tr>
<tr>
<td>Domed plate not provided</td>
<td>Unaware of its significance in inclined seams</td>
<td>8</td>
<td>Jugad approach</td>
<td>8</td>
<td>Strata mgmt cell at area to guide</td>
<td>9</td>
<td>576</td>
<td>Training and dissemination with facilitation, involvement of research institute</td>
</tr>
<tr>
<td>Appropriate wrench for tightening bolts, missing</td>
<td>Wrench not available</td>
<td>7</td>
<td>No detailed training</td>
<td>7</td>
<td>Mech fitter/helper, stores etc.</td>
<td>6</td>
<td>294</td>
<td>Tools be provided/ surprise check, make available spare wrench etc</td>
</tr>
<tr>
<td>Soaking of capsules improper</td>
<td>Unaware of its importance in appropriate manner</td>
<td>8</td>
<td>Proper jugad of soaking not provided at mine</td>
<td>8</td>
<td>Some arrangement of soaking remains at mine</td>
<td>8</td>
<td>512</td>
<td>Graduate civil engineer to be in strata management cell of mine and should visit faces of roof bolting regularly and guide accordingly</td>
</tr>
</tbody>
</table>
NEURAL NETWORK OR EMPIRICAL CRITERIA?
A COMPARATIVE APPROACH IN EVALUATING GROUND VIBRATION IN KAROUE - 3 UNDERGROUND CAVERN-SW IRAN

S MF Hossaini¹, A. Alipour¹ and A. Jafari¹

ABSTRACT: In this study results of an investigation into ground vibrations of an underground excavation in south-west Iran has been discussed. Recorded experimental blast data have been analyzed employing two different methods of analysis. A comparison between the two ways of investigation, namely empirical equations and neural network, is presented. It has been shown that the applicability of neural network method is, by far, more promising than any of three selected empirical equations. It was also found that, in spite of releasing high correlation of determination ($R^2$), empirical equations face discrepancies with real data in high range of vibration intensity whereas neural network fit the data of all ranges with a consistent accuracy.

INTRODUCTION
The hydroelectric project of Karoune-3 requires 3.3 and 1.9 million cubic meter of surface and underground excavation respectively. Drilling and blasting is the technique selected for excavation in this site. Therefore, optimization of the blast operation is of the most importance.

In a large power plant project such as Karoune -3, in most occasions, blasting is performed in the vicinity of underground spaces, structural foundations, monitoring equipments and site machinery like turbines and electrical generators. Therefore, restrictions have to be imposed on ground vibration intensity to avoid any damages to the surrounding facilities.

To come out with proper amounts of maximum instantaneous charge which produces limited ground vibration, several empirical equations are available in literature (Jimeno, C L and Jimeno, E L, 1995 and Dowding, C H, 1996). These empirical equations are normally used for estimating peak particle velocity (ppv) of ground vibration by blasting.

In recent decades artificial neural networks (ANN’s) has emerged as a powerful tool for rock engineering analysis. As a branch of artificial intelligence, ANN’s have got the ability of calculating some logical functions in forms of non-linear analyzers. In this paper, both conventional empirical criteria and ANN’s method have been used in predicting ground vibration. A comparison between the applicability of the two methods has been demonstrated.

SITE DESCRIPTION
Karoune-3 dam and power plant is located in east of the town of Izeh in Khouzestan province south west of Iran. This project is the largest amongst many of its kinds constructed or under constructions nationwide. Underground excavations of this site including power plant spaces, tunnels, water passages and drifts are located adjacent to the main body of the dam. Geologically, the project site consisted of limestone, marne-limestone, marlstone and shale. Table 1 summarizes the main mechanical properties of the site. The outline of drilling and blasting pattern at Karoune -3 was as appears in Table2.

Table 1 - Mechanical properties of the site.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Specific gravity $/m^3$</th>
<th>Yong modulus (GPa)</th>
<th>UCS (MPa)</th>
<th>RMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>limestone</td>
<td>2.5</td>
<td>20</td>
<td>98</td>
<td>75</td>
</tr>
<tr>
<td>marne limestone</td>
<td>2.4</td>
<td>20</td>
<td>98</td>
<td>65</td>
</tr>
<tr>
<td>marlstone</td>
<td>2.3</td>
<td>17</td>
<td>50</td>
<td>55</td>
</tr>
<tr>
<td>shale</td>
<td>2.3</td>
<td>17</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

¹ Faculty of Mining Engineering, University of Tehran, Tehran, Iran
Table 2 - Drilling and blasting outlines.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Floor blasting</th>
<th>Tunnels</th>
<th>Drifts</th>
</tr>
</thead>
<tbody>
<tr>
<td>hole diameter (mm)</td>
<td>64</td>
<td>45&amp;51</td>
<td>32</td>
</tr>
<tr>
<td>hole length (m)</td>
<td>3-3.6</td>
<td>2-4</td>
<td>1.2-3</td>
</tr>
<tr>
<td>dynamite cartridge diameter (mm)</td>
<td>30</td>
<td>22&amp;30</td>
<td>22</td>
</tr>
<tr>
<td>powder factor (kg/m³)</td>
<td>0.25</td>
<td>0.8-2</td>
<td>2</td>
</tr>
</tbody>
</table>

VIBRATION ESTIMATION BY EMPIRICAL EQUATIONS

To modify and improve drilling and blasting pattern of underground excavations, ground vibration was monitored during the operations. Using three selected empirical equations (Hossaini ans Sen, 2004) the data then were analyzed. These three equations are versions of the following general form of all of these types being used by investigators (Hossaini and Sen, 2006):

$$PPV = v = k R^a Q^b$$

Where $v$ is peak particle velocity in mm/s, $R$ is distance in meter, $Q$ is the maximum instantaneous amount of explosive charge in kg and $k$, $a$ and $b$ are normally called site specific parameters.

Table 3 and Figure 1 represent the results of applying the equations to the ground vibration data where comparison between the applicability of the criteria is available. As seen in Table 3 that equation 3 is the best fit to the data, with greater coefficient of determination ($R^2$).

Table 3-Drilling and blast outlines

<table>
<thead>
<tr>
<th>Equations</th>
<th>$K$</th>
<th>$B$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$PPV = K \left[ \frac{R}{\sqrt[2]{Q}} \right]^b$</td>
<td>1825.1</td>
<td>2.435</td>
<td>0.81</td>
</tr>
<tr>
<td>$PPV = K \left[ \frac{R}{\sqrt[2]{Q}} \right]^b$</td>
<td>7671.6</td>
<td>2.415</td>
<td>0.77</td>
</tr>
<tr>
<td>$PPV = K \left[ \frac{Q}{\sqrt[2]{R^2}} \right]^g$</td>
<td>400.86</td>
<td>2.232</td>
<td>0.84</td>
</tr>
</tbody>
</table>

NEURAL NETWORK

Artificial neural networks (ANN’s), are massively parallel, distributed and adaptive systems, modeled on the general features of biological networks with the potential for ever improving performance through a dynamical learning process. Neural networks are made up of a great number of individual processing elements, the neurons, which perform simple tasks. A neuron is the basic building block of neural network technology which performs a nonlinear transformation of the weighted sum of the incoming inputs to produce the output of the neuron. The input to a neuron can come from other neurons or from outside the network. The nonlinear transfer function can be a threshold, a sigmoid, a sine or a hyperbolic tangent function (Samui, P and Kumar, B, 2006).

Neural networks are comprised of a great number of interconnected neurons. There exists a wide range of network architectures. The choice of the architecture depends upon the task to be performed. For the modeling of physical systems, a feed forward layered is usually used. It consists of a layer of input neurons, a layer of output neurons and one or more hidden layers. In the present work, a three-layer feed forward network was used.
To establish an optimal network, one needs to begin with training and testing the artificial neural networks using a subset of all data sets. This process is referred to as a pilot experiment. This experiment is based on a certain number of samples; a sample being a set of input data and observed/measured information. In the pilot experiment data set, the samples are divided into a training set and a validation set. Networks with different numbers of hidden nodes will be trained all the way to the convergence of the training samples, measuring their performance with the validation set, and choosing the network that yields the best performance of the validation set. Finally, this selected network model will be used for the whole data set.

Performance of the developed network was tested with the help of:

(i) drawing a scatter diagram of estimated versus target values.
(ii) Computing mean absolute error (MAE) using:

\[ MAE = \frac{1}{Q} \sum_{i=1}^{Q} |y - x| \]  

(4)

Where;

- \(x\) is target
- \(y\) is network output
- \(Q\) is number of test patterns.

(iv) Computing mean square error (MSE) using:

\[ MSE = \frac{1}{Q} \sum_{i=1}^{Q} (y - x)^2 \]  

(5)

Figure 1 - Peak particle velocity versus scaled distance for the three equations
Where:

\( x \) is target
\( y \) is network output
\( Q \) is number of test patterns

VIBRATION ESTIMATION BY NEURAL NETWORK

When supplied with adequate vibration data neural networks are capable of drawing a relationship between peak particle velocity from one hand and distance and maximum instantaneous charge from the other hand. Distance and maximum instantaneous charge are introduced as inputs of the neural network as appears in Figure 2 for the data of Karoune-3. An optimized model of neural network built after several executions in MATLAB environment is detailed in Table 4. Coefficient of determination \((R^2)\) between real and estimated values of ppv for training and testing groups are 0.98 and .94 respectively.

![Figure 2 - Neural network model used for analyzing Karoune-3 data](image)

COMPARISON BETWEEN APPLICABILITY OF THE TWO METHODS

As far as the comparison between the applicability of the empirical equations is concerned Different results can be achieved by applying the same empirical criterion to different cases. An empirical criterion which offers an outstanding correlation with a group of data may lead to discrepancy in another case of the same nature. Therefore, specific equations have to be found for specific cases. The results of applying the three empirical equations and neural network are compared in Table 5. As seen in this Table, the applicability of neural network is by far better than any of the equations.

Figure 3 visualizes the degree of agreement of the equations as well as neural network model with the data. In this figure, the data are sorted and numbered in ascending order of ppv.

As far as the comparison between the applicability of the empirical equations is concerned, Equation 3 provides better \(R^2\) (Table 3) while Equation 1 is better in MSE and MAE factors (Table 5). Discrepancy of the three equations with data of higher range of ppv (over 20 mm/s) is observed in Figure 3 whereas neural network is quite promising in that data range too. These discrepancies are the reason for getting different outputs when different statistical methods are employed. This implies that coefficient of correlation\((R)\) or coefficient of determination \((R^2)\) are not lonely capable of being a proper tool of judgment over the whole range of a data groups.
Table 4 - Details of optimized neural networks model built for Karoune 3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Related information</th>
<th>Parameter</th>
<th>Related information</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of train data</td>
<td>26</td>
<td>Global error function</td>
<td>MSE</td>
</tr>
<tr>
<td>No. of test data</td>
<td>7</td>
<td>No. of optimum neurons in hidden layer</td>
<td>30</td>
</tr>
<tr>
<td>ANNs Structure</td>
<td>2-30-1</td>
<td>No. of optimum epochs</td>
<td>35</td>
</tr>
<tr>
<td>Activation function of hidden layer</td>
<td>Log_Sig</td>
<td>MAE for train and test data</td>
<td>20.06, 1.93</td>
</tr>
<tr>
<td>Activation function of output layer</td>
<td>Linear</td>
<td>MSE for train and test data</td>
<td>8.77, 13.02</td>
</tr>
<tr>
<td>Train algorithm</td>
<td>Levenberg_Marquardt</td>
<td></td>
<td>_</td>
</tr>
</tbody>
</table>

Table 5 - Comparison of error values in various approaches.

<table>
<thead>
<tr>
<th>Model</th>
<th>MAE</th>
<th>MSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation 1</td>
<td>8.97</td>
<td>213.80</td>
</tr>
<tr>
<td>Equation 2</td>
<td>9.38</td>
<td>221.82</td>
</tr>
<tr>
<td>Equation 3</td>
<td>9.33</td>
<td>361.53</td>
</tr>
<tr>
<td>Neural network</td>
<td>3.31</td>
<td>27.43</td>
</tr>
</tbody>
</table>

The problem of different applicability in different ranges of data does not happen in the case of neural network method. As Figure 3 explains, this method is in very good agreement with the data in all ranges. This can be regarded as a prime advantage of this method.

CONCLUSIONS

- Empirical criteria for ground vibration evaluation may submit different results in different cases or even in different ranges of data of the same case.
- Although having lower rate of agreement with real data comparing to neural network method, empirical criteria, due to their simplicity in usage, can be regarded as a useful tool provided that specific equations are searched for specific range of any specific case.
- Neural network is a powerful, precise and reliable tool of predicting ground vibration having excellent results in different data ranges yielding much better results than any empirical equations.
REFERENCES

AN APPROACH TO ADDRESSING EXPLOSIVE RELATED ACCIDENTS BY IMPLEMENTING STRATEGIC TRAINING

Gour C Sen and Geoff Downs

ABSTRACT: In underground coal mines, the use of explosives is very limited, particularly where the longwall method of mining is practised. However, occasionally the call for the use of explosives does occur in most underground coal mines. Explosive use today is generally confined to shooting a dyke, excavating an overcast in stone, for fault drivage, and for shaft sinking or drifting in cases where use of a road header is considered to be uneconomical. Explosives have also been successfully used in triggering goaf falls to prevent wind-blast or for remote mining through areas where methane gas does not drain below the 'outburst threshold limit'. It is therefore essential that among the mine personnel there are employees qualified in the handling of explosives under varying conditions, and these qualifications should be up to date.

This paper proposes that all shotfirers should undergo a refresher course in blasting at intervals of five years, so that up-to-date knowledge is disseminated. The essential modules of such a refresher blasting course should consist of the following: a) Familiarisation with current types of explosives and accessories; b) Types and modes of initiation; c) Blast design and calculation; d) Precautions to be taken before and after firing; e) Consideration of environmental factors; and f) Dealing with misfires. Above all, a safety culture amongst the personnel who handle explosives needs to be established, and strictly adhered to.

INTRODUCTION

In mining and in civil engineering projects, blasting is one of the most potentially hazardous operations. It is recorded (Sen & Hayward, 2007) that blasting related accidents have claimed the lives of more than a thousand people around the world since the turn of the Millennium. Most of these accidents are due to flyrock and an inadequate blast exclusion zone. A number of recent explosive related accidents will be cited and conclusions drawn. Explosives manufacturers are continually developing their product, with improved safety characteristics, and it is fair to comment that most blasting accidents are usually due to human error or ignorance, rather than a faulty product. The impact of increasing safety and environmental legislation as well as the growing explosives security requirements demand that blasting operations should be managed by shot-firers who are competent in all facets of explosives usage. This is particularly pertinent to those shot-firers who get sporadic, often very infrequent calls to undertake blasting operations.

Many European countries (Akhavan et al, 2007) are endeavouring to review explosives regulations and realise that training with respect to the management of blasts needs to be improved. Apart from primary risk associated with the storage, transport and handling of explosives, there are possible operational risks due to flyrock or misfires caused by poor blast design. The major shift is to train the blasting crews so that the new training scheme prepares them for competence-based, rather than knowledge based qualifications.

This paper firstly deals with some explosives related incidents and analyses their causes. This will be followed by instructions in line with the directives of the Australian Forum of Explosives Regulators – a body seeking to harmonise explosives regulation throughout Australia – training with respect to the management of blasts needs to be improved.

EXPLOSIVES RELATED INCIDENTS

Numerous explosives related accidents have occurred, and it is not possible to mention or analyse them here. This paper will endeavour to focus on certain incidents from which conclusions can be drawn.

Porgera Gold Mine, Papua New Guinea Explosion

This explosion totally destroyed the explosives manufacturing facility on the surface in August 1994 with the loss of 11 personnel (Sen and Abrahams, 2005). There were two explosions, about 1 hour 15 minutes apart. The fires on the bulk truck around and underneath the bulk emulsion and process oil tanks subsequently caused the second explosion, which was larger than the previous one. It is estimated that approximately 5½ tonnes of various types of explosives were destroyed during the first explosion, creating a crater of 7 x 5 x 2 m. The second explosion

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2 Chief Inspector of Explosives, Dept. of Mines & Energy, Queensland
produced a crater of approx. 40m in diameter and 15m deep, with the destruction of nearly 90 tonnes of explosives.

After extensive analytical work, it was found that the first explosion occurred in the emulsion explosives packaging line, specifically within the Mono pump. Although the exact mechanism of the initiation of the explosion is not known, it is believed that the initiation point for the first explosion took place inside the rotor boss-head cavity, where the emulsion explosives compound managed to leach pass O-rings and a possibly missing packing gland over a period of time. It is thought that microscopic cracks, flexing of components, heat and possibly back pressure were sufficient to create the right conditions to initiate the explosive compound in the cavity. Some emulsion explosives exhibit strong exothermic reaction above 230° C.

A planned inspection programme covering critical plant and equipment, parts and items could have avoided this massive destruction.

**Incident during demolition of buildings, Canberra, Australia**

This incident happened in July 1997 in the course of the demolition of a tower block constructed of reinforced concrete with steel columns. The building, to be demolished by implosion, was situated on the edge of a lake. An estimated 30, – 40,000 spectators were gathered around the lake on the far side of the building. A young girl, who was standing about 430 m from the explosion site watching the demolition, was hit by a flying projectile and died instantly. Her death was caused by a fragment of steel expelled from one of the corner columns of the building. This steel projectile, which weighed nearly 1 kg, struck the girl’s head. This impact caused the girl’s scalp and skullcap to sever from her head.

The impact velocity of the steel fragment was estimated to be 128 – 130 m⁻¹, with an associated energy of around 8.172 kJ. A piece of fractured steel from another backing plate was embedded in the grounds of a house about 400 m from the blast site. The plate was warm to the touch. Other metal debris, recovered from the surrounding area after the blast showed the same qualitative characteristics that generally occur when steel is intimately exposed to a sudden explosive impact.

Subsequent enquiry revealed that there were a large number of weaknesses in executing the project, the prominent ones being: a) the blasting contractor did not have sufficient experience in imploding a large building; b) there was a lack of pre-weakening of some pertinent structure of the building; c) the quantities of explosives used were clearly excessive; d) the protective measures were inadequate; e) there was an inappropriate exclusion zone to ensure the safety of the crowd and f) the height of the bund erected for containing projectiles was insufficient.

**Truck explosion, Hunter Valley, Australia**

A utility truck, used for carrying explosives in an open cut coal mine, exploded in February 2003. After loading 97 holes with boosters and detonators for the blast of the day there remained a surplus of one box containing 23 detonators, two full boxes of boosters and one box containing 38 boosters. The detonator box and the partly filled booster box were placed at the rear right of the truck, and two full booster boxes on left side of the tray.

The shotfirer then had liquid refreshment and smoke, after which he went to the ‘crib hut’ and took a packet of cigarettes and placed it in the cabin of the truck. Then he drove the vehicle to the magazine gate, which he stopped to open. After getting out of the truck, the shotfirer noticed smoke coming from the driver’s side of the rear tray. He ran to the passenger side at the front of the truck and crouched down, when the vehicle exploded. The shotfirer was hit by flying gravel and knocked over, clear of the moving truck.

The locked gate restricted forward movement of the truck, but was smashed open by the post-exploded vehicle. The dazed shotfirer regained his feet and wandered around the compound in the direction of the ‘crib hut’. Subsequently he was found lying on the ground outside the hut.

The injured shotfirer was taken to the hospital to be examined, but apart from small cuts, abrasions, bruises to arms and legs he was not seriously injured. The shotfirer must have been shielded from the explosive force by the cabin, and his injuries were caused by the shock waves from under the truck. Eardrum damage would have normally occurred within 15m of the explosion and although he was actually within a 10m limit, because of the shielding effect of his actual position, he sustained no increased injuries. If the shotfirer had been in a standing position in front of the truck, the windscreen would have decapitated him.

It was clear that the cause of the whole incident was that the shotfirer flicked a burning cigarette out of his window whilst driving, and it landed on a detonator box. The movement of the air over the cigarette caused the box to ignite, which in turn ignited the foil and finally the detonators. The detonators then set off the open box of boosters, which in turn set off two unopened boxes of boosters almost immediately.
Some snapshots of incidents around the World (Anon)

**Unplanned detonation of seismic detonators, South America**

This incident, involving electric seismic detonators, occurred in South America and resulted in the death of one operator and the amazing survival of another. The survivor has been able to give a complete description of the events leading up to the accident. From this it has been possible to eliminate the obvious causes, such as radios and batteries etc.

The essential details are that a primer charge of Pentolite was being prepared by taping the wires of the detonators to the outside of the charge, when it detonated. The primer charge was a typical plastic seismic cartridge, and two cartridges were joined together, each with a detonator in the detonator well. The detonator leads were shorted and the bared wires insulated. The weather was about 20°C and not particularly conducive to static generation. The operator was wearing light clothing and was kneeling on a wooden platform taping up the primer when it exploded.

The opinion is that an electrostatic discharge must have caused the incident. Technically, the plastic tape being unwound generated a voltage, so that a charge was induced on the detonator. It is postulated that a spark occurred between the shell of the detonator and the fusehead assembly and leadwires.

**Explosion while burning empty explosive boxes**

After loading a face of a quarry with explosives and initiating devices, a shot-firer collected the empty boxes ready for disposal by burning, and liberally sprinkled diesel fuel to aid combustion. As it was a cold day, he decided to warm himself by the fire. Suddenly there was an explosion, and the man was fatally injured by the blast.

The subsequent investigation revealed that during the shot loading operation, someone had placed an off-cut of heavy duty detonating cord into one of the boxes for later disposal and failed to warn anyone of this fact. The burning of the empty cases, aided by the diesel oil, led to the explosion of the detonating cord. Further, the shotfirer had been standing too close to the fire. Had the shotfirer obeyed basic rules for burning discarded explosives packaging, this accident could have been avoided.

**Lightning initiated an explosion in a surface mine, Australia**

In 1968 lightning hit a totally non-electric blasting work area in a surface mine in Western Australia. All the blastholes were loaded and were to be initiated with detonating cord downlines and mainlines, and then to be fired with a capped safety fuse.

A rain-storm with associated lightning moved into the area. Because it was hot, the blast crew decided to stay out in the rain to cool off and continued to charge the holes with gelignite. The reel of detonating cord was located on a vertical stake at one end of the blast site. Downlines for each hole were cut off this reel as the blast-crew moved along the row, and eventually the detonating cord lay over the tops of a number of charged holes.

The lightning got close and finally hit the blast site, when the gelignite detonated. This initiated the detonating cord back to the reel, which promptly exploded violently. The two men who were charging were killed. Random holes along the face were initiated where the cord lay over the ends of the downlines.

Safe practice dictates that all shot firing operations should cease in the vicinity of an electrical storm. Moreover, a mainline detonating cord should never be casually run over the downlines. The reel should be taken to each hole and then cut leaving an adequate length secured at the collar. When all the holes are charged, only then is the reel run out to form a mainline. This practice minimises the time that the mainline cord is actually connected to the holes.

**Explosion in a truck carrying detonators, China**

A truck carrying 285,000 blasting caps exploded in the southwest city of Chongqing, killing 14 persons and injuring 48. Explosions from this blast lasted almost half an hour, blowing out windows in a 50m radius. The driver of the truck and a passenger were killed, while other victims were nearby pedestrians.

It is suspected that a burning cigarette came in contact with the cardboard detonator box, which caused a small fire, followed by the explosion.

**Explosion in an underwater blasting operation, Newcastle, Australia, 1978 (Apel, 2005)**

The aim of this blasting operation was to deepen the harbour channel to allow larger vessels to berth. The work involved drilling and blasting from a barge where the water depth was between 15 to 20 m. Nitroglycerine based explosives (AN60) with detonating cord were used. Occasionally, drilling rods were used as a tamping rod to push the explosive cartridges into the blasthole.

One day, two lengths of drilling rods were being mechanically joined on the barge surface and caused an explosion in which two workmen were killed.

Subsequent investigation revealed the cause of the accident. Since drilling rods were used to tamp the explosive cartridges, some parts of the explosive cartridge accumulated in the rod threads as well as in the hollow section of
the rods. This explosive residue was not visible, as the rods were partially covered with mud. The pressure created by the mechanical joining of the rods initiated the explosion. This is a salutary example of the consequences of using an inappropriate tool.

Incidents with trucks carrying explosives, Spain
Two incidents are quoted here. The first one happened in Castellon, Spain, when a lorry carrying ammonium nitrate collided head-on with two cars, killing two persons and injuring three. The damaged heavy goods vehicle overturned and left the road. The diesel fuel from the broken tank came in contact with the load of ammonium nitrate, which initiated an explosion half an hour after the vehicle overturned. The resulting explosion could be heard in villages more than 15 kilometres away. The lorry was totally destroyed, leaving a crater several metres across. The fireballs generated travelled for a radius of up to one kilometre around the area of the accident, leading to 15 small fires. After this accident, the Special Risk Plan for the Carriage of Dangerous Goods was introduced.

Drilling into a charged hole, Denmark, 1995
Two men had been drilling in a rock excavation operation. After drilling a few holes and charging them with nitroglycerine based explosives with electric detonators, they found the remainder of a previous blasthole, which is often termed a ‘butt’. They started drilling into this ‘butt’ until an explosion occurred, which injured both men.

Subsequent investigation concluded that the men were drilling into un-detonated explosives. Fortunately, the firing circuit with all charged blastholes had not been joined. Of greater concern is that neither of the men had attended a blasting course and that they were using a 12V battery to initiate the shot.

Explosives truck blown up by candles, China, 1996
A truck carrying one tonne of explosives, 5,000 detonators and 500 metres of detonating cord blew up in Sichuan Province. The explosives were accidentally ignited by workers when they lit candles to provide light while unloading the truck.

At least 14 people were killed, more than 30 seriously injured and an unknown number of people missing. All houses were totally destroyed within a radius of 100 metres of the blast, which occurred in an agricultural machinery station in Yuyang County, Sichuan Province.

Premature explosion of detonating cord reel
At a large open pit operation, low energy cord was being used to link-up down lines of standard cord protruding from blastholes. The surface lines were deployed from off the reel from the back of a moving truck when suddenly one of the lines become snagged and snapped. There was an explosion and the remaining detonating cord in the reel exploded into the back of the truck, seriously injuring the operators.

At the enquiry following this accident, it was concluded that the practice of unreeling the detonating cord from a moving truck had the potential to cause a serious accident if the cord becomes entangled or snagged. When this occurs, tension and friction created at the surface connection, when used, is believed to heat the priming element making it much more sensitive to initiation by friction or shock. Premature detonation can occur when the cord is under tension.

Collision of a truck loaded with ANFO, Mexico, 2007 (Abrahams, 2007)
A truck, loaded with 25 tonnes of ANFO, crashed into a pickup on a two-lane highway, and then burst into flame. It took about ¾ of an hour for the cargo to explode after the accident, and created a huge hole in the road. This accident took place near a small farming village and an aquatic recreation centre packed with families on weekend outings. The questions arose about why authorities did not evacuate the scene. The chief of police investigations in Monclova, a steel city a few miles from the accident site, said the driver of the truck and a co-driver had survived the explosion and were wanted for questioning.

STRATEGIC TRAINING PROGRAM

Introduction
This section describes the Queensland Explosives Inspectorate approach to managing explosives’ safety and security, shotfirer licensing, and reviews the proposed directions of the Australian Forum of Explosives Regulators, the work of Skills DMC: National Industry Skills Council in developing uniform national shotfirer competencies. It evaluates the need for skills maintenance of shotfirer competencies and a general evaluation of training and competencies of others in the explosives’ life cycle.

Explosives are an essential tool to the mining and some other industries. The management and control of explosives in its life cycle is essential to the health, safety, security and well being of all those involved in its lifecycle. The lifecycle includes all those activities involved in the import, export, sale, use, transport, storage, manufacture and disposal of explosives. These activities are generally licensed under explosives legislation. On a mine site, the people with a role in explosives and blasting may include -
• Drill and blast superintendent;
• Blast designer;
• Driller;
• Blasting contractor;
• Shotfirer;
• Shotfirer assistants;
• Magazine keeper;
• Vehicle drivers;
• Transporters; and
• Explosives manufacturers including mobile manufacturing units and ANFO blow loaders. Of these people, the shotfirer is one of the few people who must have mandatory competencies for an accompanying licence or equivalent.

Queensland Explosives Inspectorate Approach

The Queensland Explosives Inspectorate administers the *Explosives Act 1999* and Explosives Regulation 2003. The approach of the inspectorate is governed by the legislation and the vision and mission. The vision of the inspectorate is *Our community safe and secure from explosives* and the mission is *To protect our community from the adverse impact of explosives*.

The approach of the Explosives Inspectorate is based on the approach of Professor James Reason and the Swiss Cheese Model and is summarised in Figure 1.

![Figure 1 - Queensland Explosives Inspectorate Strategies for explosives incident prevention and risk minimisation](image-url)

Briefly, in the “Swiss Cheese” model, a series of barriers and defences are put in place to prevent incidents. Each barrier and defence is not entirely 100% effective and hence there is an opening in that barrier and defence and hence the hole. The hole may open and close due to the changing circumstances altering its effectiveness. The more barriers and defences that are effective are put in place, the more reliable and effective the strategy and system will be. When the holes in the “Swiss Cheese” line up, an incident will occur.

From Figure 1, a competent industry together with suitable education and training are important barriers and defences in the strategy to prevent explosives incidents and minimise risk. Effective training and competencies are
essential strategies. Training is a combination of formal training and on-the-job training. A similar type of model could be developed by a workplace for the development of their own strategies to prevent accidents and minimise risk.

**Shotfirers**

Now specifically looking at shotfirers in Queensland, a shotfirer for underground or above ground coal activities must be licensed under the Explosives Act 1999 or appointed as a shotfirer by the Site senior Executive under the Coal Mining Safety and Health Act 1999. A shotfirer licence is issued for a one or five year period. For a new shotfirer licence to be issued or renewed, a current Statement of Attainment from an accredited Registered Training Organisation (RTO) under the Australian Quality Training Framework (AQTF) must be provided with the application. The Statement of Attainment must be dated within the last three years and must have the required national competency units from a national training package. Table 1 in the Appendix is used by the Queensland Explosives Inspectorate to determine if the requisite competencies from the agricultural, metalliferous, construction industry, quarry or coal packages have been obtained. The columns in Table 1 provide a list of alternative competencies that can be obtained to achieve the desired category of licence, Category 1 being agricultural, seismic and small scale blasting, Category 2 is quarrying, open cut and construction and Category 3 is tunnelling and underground mining. As seen from the table, the selection of competencies is quite complex considering the competencies options from the agricultural, metalliferous, quarry or coal packages, which have equivalent competencies.

The origins of the current blasting competencies are set within a number of nationally accredited training packages under the Australian Quality Training Framework. These training packages can be found on the National Training Information Service (NTIS) website [www.ntis.gov.au](http://www.ntis.gov.au). These include military, transport, coal mining, quarrying, metalliferous mining, civil construction and agricultural training packages. For blasting, the three nationally accredited mining training packages referred above each contain units, which outline competency requirements for topics from magazine keeping to blast design.

**Skills DMC: National Industry Skills Council**

Currently the Skills DMC: National Industry Skills Council is managing a consolidation project aimed at rationalising duplication of similar competency units within several closely allied training packages. The common units should be released in late 2008, for use in metalliferous and coal mining, quarrying and construction industries. The proposed consolidation of the shotfiring units will simplify the complexity of competencies across the three nationally accredited mining training packages and this work has been the subject of consultation and review with stakeholders from government, industry and the unions who are in agreement with the proposed changes. The Skills DMC: National Industry Skills Council has kept the members of the Australian Forum of Explosives Regulators (AFER) briefed on the progress and development of the work.

Skills DMC: National Industry Skills Council is a national organisation with the primary role of facilitating the Vocational and Education Training needs of all stakeholders within the Coal, Civil Construction, Construction Materials, Drilling and Metalliferous industry sectors.

**Australian Forum of Explosives Regulators**

In addition to the consolidation of the shotfiring units by the Skills DMC: National Industry Skills Council, all States explosives regulators who are representatives at Australian Forum of Explosives Regulators (AFER) have agreed to adopt and transition towards using these nationally accredited competency units as a basis for their respective shotfirer licensing regimes.

Through the activities of the AFER, all States explosives regulators have also committed to consistent licensing and licensing criteria for all explosives related activities including shotfirers. Not only has it been agreed, but it is currently under active review. All States explosives regulators have agreed to adopt the national competencies as the basis for licensing.

To date, there has not been discussion or agreement at AFER regarding the refreshing of competencies at regular intervals e.g. every five years.

National competencies for shotfirers have existed for many years now. Not all jurisdictions require evidence of the national competencies for licensing purposes. Even though this position exists, all jurisdictions at AFER have committed to their adoption.

The AFER is the forum of government authorities responsible for administering explosives safety and security legislation in Australia. The AFER reports to the Workplace Relations Ministers' Council (WRMC) on the development of nationally consistent explosives regulation, through the Australian Safety and Compensation Council (ASCC). Membership of AFER consists of two representatives from each Australian state and territory, Australian Government including Defence, Civil Aviation Safety Authority (CASA), and Australian Maritime Safety Authority (AMSA).
AFER's terms of reference include but are not limited to:

- acting as the lead body to provide recommendations to governments through the WRMC on nationally consistent explosives regulation;
- promote the development and implementation of nationally consistent legislation and safety and security standards to Ministers, heads of agencies and associated parties.

**Maintenance of competencies**

In Queensland, a person applying for a new or renewed shotfirer licence must provide a Statement of Attainment dated within the last three years for the required national competencies provided by an approved Registered Training Organisation. This position is at variance with the State and Territory bodies administering the AQTF. Their assertion is that a person who is deemed competent will remain competent and that competency does not expire.

The approach of ensuring that a Statement of attainment is dated within a certain period of time e.g. three or five years was introduced in the Explosives Regulation 2003 in Queensland. A review of training and competencies was undertaken as a part of the investigation into the fireworks tragedy at Bray Park Brisbane in May 2000. Franklin (Franklin 2001) in his Training and Competencies Report recommended that a program should be developed that states requirements for skills maintenance of all licence holders to the initial standards. Franklin (Franklin 2001) also recommended that legislation must be developed that requires both formal and on-the-job training as an initial licensing requirement, and on-going refresher training as a condition of licence holding.

It is recognised that fireworks operators had similarities to underground coal shotfirers to the extent that they conduct their explosives activities on an infrequent basis.

In addition, from reviewing explosive incidents and complaints undertaken by the Queensland Explosives Inspectorate, the knowledge and skills of shotfirers has been contributing factors. Within the last 15 years, there have been great changes in most areas within the explosives life cycle. There had been the implementation and adoption of new legislation, Explosives Act 1999 and Explosives Regulation 2003, changes for control and security of security sensitive ammonium nitrate, new and updated Australian Standards AS2187 Parts 1 and 2 for the storage and use of explosives, new code for the transport of explosives by road and rail, new industry codes for the explosives precursors, mobile processing plants and hot and reactive grounds. The Explosives Act 1999, like the Coal Mining Safety and Health Act 1999 is performance based adopting a risk management approach to the management of health and safety.

During these periods, technology has advanced with the implementation of new initiation systems, explosives companies introducing changed products including safer explosives and with the growth of the industry, shots are being larger. With these changes have also come new challenges such as hot and reactive ground in areas where they were unknown for the new products being used. In general, there was considerable change. This has driven the mechanism to be in place for the shotfirer to upgrade their skills. This mechanism is the refreshing of the competencies though possessing a current Statement of Attainment. The issue date should be within the last three years.

During this period, the world security crisis has resulted in the upgraded focus on security of explosives, ammonium nitrate and explosives precursors and the appropriateness of people having supervised and unsupervised access.

In addition to these technical changes, the mining industries are facing huge challenges with skill shortages, high staff turnover rates and loss of corporate memory. In the Australian Mining Journal Nov/Dec 2005, the Newcrest Managing Director reported that 41% of the employees of that company had been with the company for less than one year.

All shotfirers must have and maintain a desirable level of competence. To achieve this, shotfirers must be initially well trained. There is sufficient data (Franklin 2001) to suggest this can be achieved in a structured training course, which includes sufficient task skills development. There is also sufficient evidence (Franklin 2001) to suggest this is more likely to occur in a formal training environment than by potentially ad hoc on-the-job training. Data (Franklin 2001) has shown that initial competence can be lost over time and/or replaced with alternative approaches. This indicates that competency can be lost over time. One explanation is the loss of theoretical knowledge in relation to safety matters. Other explanations are that practical skills were never gained, such as with limited practical training or were lost by lack of use due to limited numbers of blasts. A combination of the two approaches is another obvious explanation.

Under legislation, a person must be trained and competent for the role and tasks that they will be undertaking. In addition to the identification of formal training needs, all workplaces managing explosives must develop operate and maintain a safety management system and security plans. These activities require, in addition to formal training to national competencies, on the job training and for this training to be refreshed.
There is a combination of formal training and on the job training for all jobs. The greater the reliance and use of national competencies is well recognised and supported. The relative timing of formal training and on-the-job requirement is important.

The essential modules of such a refresher blasting course should consist of the following:

- Familiarisation with current types of explosives and accessories;
- Types and modes of initiation;
- Blast design and calculation;
- Precautions to be taken before and after firing; and
- Consideration of environmental factors; and
- Dealing with misfires.

This is essentially renewing the national competency which can be done by a combination of course work and recognition of prior learning. The addition of reviewing accidents and incidents such as those identified earlier in this paper would be most useful.

As seen above, national competencies exist for shotfirers and these are being rationalised by the Skills DMC: National Industry Skills Council. It is also noticed that not all states explosives regulators require that shotfirers must be competent under these national competencies. Because shotfirers must hold licences or equivalents based on these competencies in Queensland, there are eight Registered Training Organisations in Queensland approved to deliver training and refresher for shotfirers to these national mining competencies covering explosives. This training covers shotfirers, assistant shotfirers and magazine keepers. This is a very effective and satisfactory outcome.

**European developments**

A recent paper (Akhavan et al, 2007) describes developments in the European Qualification for Workers in the Explosives Sector. Over the past three years, the UK, Sweden, Norway, Finland and Italy have taken part in a program under the EU funded Leonardo Da Vinci program. The UK has developed National Occupational Standards for the civil and defence sectors. The National Vocational Qualifications define the knowledge and skills and how these form the basis of vocational education. Currently there are 440 National Occupational Standards and competencies, 24 National Vocational Qualifications within 12 key roles. These can be found on [http://www.qca.org.uk/610.html](http://www.qca.org.uk/610.html) under explosives substance and articles. People have the ability to subscribe to some of these competencies over the web for a fee.

**CONCLUSION**

The developments in consolidating and rationalising the current blasting competencies by the Skills DMC: National Industry Skills Council is positive step forward for the training and competencies of shotfirers.

The commitment of the state explosives regulators to move to adopt these competencies for licensing is a positive step forward together with the step toward consistent licensing and licensing criteria.

The maintenance of the knowledge and skills of shotfirers should be refreshed at a minimum of every 5 years and this should be done through the maintaining of the national competency standard. This follows a period of rapid change in technology, legislation, standards, codes and security awareness.

**ACKNOWLEDGEMENTS**

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**REFERENCES**

Apel, D., Personal communication, 2005.
### Competency Units

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<tr>
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<td>MNMG 412A - Initiate blasts (UG)</td>
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<td>MNMG 353A - Fire Surface Blasts</td>
<td>MNM 05</td>
<td>x</td>
</tr>
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<td>MNMG 210A - Store, handle and transport explosives</td>
<td>MNM 05</td>
<td>x</td>
</tr>
<tr>
<td>MNMG 311A - Conduct secondary blasting</td>
<td>MNM 05</td>
<td>x</td>
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<td>MNMG 349 - Conduct accretion firing</td>
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### Conditions

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- **G**: 2, 5
- **H**: 2, 5
- **I**: 2, 6, 7
- **J**: 2, 8

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**APPENDIX - Table 1 – Shotfirer Competency Requirements – Queensland December 2007**
APPLICATION OF FINANCIAL RISK ANALYSIS FOR PROJECT EVALUATION AT A LARGE COAL MINE

Mohammad Zare\textsuperscript{1}, Farhang Sereshki\textsuperscript{1} and Naj Aziz\textsuperscript{2}

ABSTRACT: Today risk analysis is largely employed in studying the uncertainty of financial decisions in new mining ventures. The methodology for examination and quantification of financial risks is investigated in this paper. A coal mining project is generally a long term process. Also, any increase in the project time, raises the probability of variation in effective financial parameters. Therefore, the forecasting and risk analysis for this kind of projects will be more important. For this reason, this work has been prepared for the purpose of presenting the methodology and uses of techniques applied in the evaluation of the long term projects to analyse and assess risk. The flexibility of spreadsheets and their statistical capabilities make them a common framework for simulation modelling. In this paper, the discussion is illustrated with an example of stochastic discounted cash flow for a real large coal mine. Then, Excel spreadsheet tools for the simulation, and also a powerful risk analysis software have been used. Finally, the paper examines output data from analysis of mentioned project.

INTRODUCTION

When uncertainty and risk are relatively absent, project evaluation is a reasonably easy exercise. However, mineral projects often involve commodities for which prices or operating procedures are difficult to forecast, and analysts must make decisions concerning how best to evaluate such projects. The introduction of risk greatly complicates the evaluation process. The key to successful evaluation is the proper inclusion and quantification of risk and the consequences (Glickman and Gough, 1990).

A complete understanding of the risks, their consequences, and their probabilities of occurrence is an absolute necessity for any project evaluation. A pro forma cash flow can provide the basis for risk assessment in an investment analysis, as well as a means to determine the optimal assignment of risk in a financial analysis. The goal of risk assessment is not to reduce risk--that cannot be done by analysis--but rather to increase the understanding of risk so that appropriate action can be taken. In this paper, a large coal mining project is investigated for its financial risk analysis.

Importance of Risk Analysis in Mineral Projects

Mineral investments can have a number of characteristics that make them somewhat different from other types of investment opportunities, including the depletable and often unique nature of the ore reserves, the unique location and characteristics of the deposit, the existence of geologic uncertainties, the length of time required to place a mineral property into production, the usually long-lived nature of the operation itself, and the pronounced cyclical nature of mineral prices. In addition, mineral deposits can be mined only where the minerals are found. Therefore, options for locating a mineral operating site are likely to be more limited than with other types of industrial developments. Among the mineral projects, coal mines, especially, are more remarkable and significant. This decrease in flexibility increases the risk of mining ventures compared to other types of investment opportunities (Torries, 1998).

Each mineral deposit is unique in terms of location and ore grade and characteristics. It theoretically cannot be replaced when depleted, although similar deposits can often be found or purchased. This aspect makes it difficult to compare the value of one mineral operation with another. Risk is increased since there is no guarantee that a search will yield a new mineral deposit to replace a depleted deposit.

The effects of time greatly influence the value of a mineral project, as they do any other long-lived investment. Usually prices and costs must be predicted, which introduces an element of risk common to most other types of investment opportunities. However, many mineral prices are cyclical in nature, and the difficulty in forecasting prices and costs poses particular problems in evaluating and planning mineral projects (Labys, 1992).

Time also affects mineral projects in several ways that are not always present in other investment opportunities. First, significant mineral reserves are usually required for a long-lived mineral project. Since, by definition, reserves are quantities of ore to be mined in the future, the present value of a ton of reserves is less than the present value of a ton of ore that can be mined immediately. Many other investment decisions involve deciding whether to sell an item today or to wait until the future; thus, public officials who wish to tax mineral reserves to raise funds for public projects often misunderstand the fact that not all reserves have the same present value.

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Second, the mineral evaluation process must consider that operating decisions made early during the life of a mineral deposit will affect the long-run value of the operation. For example, early mining of higher-grade ore increases early profits but lowers the average grade of the remaining ore and reduces the life of the mine. In addition, economic and operating conditions can unexpectedly change during the life of a mine, making flexibility of operation desirable.

Last, it is impossible to know the exact amount or grade of material to be mined until the deposit is depleted, for two reasons. The first has to do with geologic certainty. The geologic quantity and quality of the deposit must be determined by sampling, which by definition gives statistical estimates. The second reason involves economic certainty. The quantity and quality to be mined at any given time depend upon mining and processing costs and the prices of the resulting commodities. Since future prices cannot be forecast accurately, it is usually difficult to determine reserves even though there may be a high degree of confidence in the geologic-based estimates (Torries, 1998).

**SIMULATION**

Simulation is a powerful and flexible analytic tool that can be used to study a wide variety of management models. It is generally used in cases where a good analytic model either does not exist or is too complicated to solve. Simulation can also demonstrate the effects of variability and initial conditions, as well as the length of time needed to reach steady state. This description encompasses a large segment of real world models and, as a result, surveys of the use of management science techniques typically put simulation at or near the top. However, there are a number of important factors for management to consider before making a commitment to a simulation study. The ability of simulation models to deal with complexity, capture the variability of performance measures, and reproduce short-run behaviour make simulation a powerful tool.

**Monte Carlo Simulation Technique**

Monte Carlo simulation method tries to investigate stochastic permutations of uncertainties, which occur in a project. The process speed and power of a computer is implemented in order to search different modes of these uncertainties. Firstly, the most proper distribution function is determined for each uncertainty found in the second phase of risk analysis process. These distribution functions are determined considering the experts' opinion and available records obtained in the previous projects. For example, total cost may have a normal distribution function with the parameters $m$ and $s^2$. Then the number of runs, that the simulation should be performed, is determined regarding the project size and the importance of risks. The number of runs can be set as 1000, 2000, 5000, and so on. It could be said that while the number of runs increases, much more stochastic modes are searched in the solution space (Pindred, 1995).

In each run, a stochastic value is allocated to each uncertainty in the range of its lower bound and upper bound. The frequency of each value is followed by determined distribution function. Therefore, a set of variables are allocated to all uncertainties in each run and the underlying utility amount is determined based on the value of these variables (Stermole, 1993).

![Figure 1 - Monte Carlo Sampling Method](image)

Application in Cash Flow Probabilistic Analysis

Probabilistic analysis can be thought of as the ultimate form of scenario analysis in that all possible cases are considered simultaneously. The input for the analysis consists of a distribution of values for each variable in a cash flow analysis. In other words, for each variable used in a cash flow, a range of values and their probabilities of occurrence are used as inputs instead of a single value as in scenario analysis. Since inputs are probabilistic, most
of the risk inherent in the project is reflected in the range of input variables. Therefore, the discount rate used in probabilistic evaluation methods must reflect the risk captured in the cash flow itself. The risk component in a risk-adjusted discount rate decreases in proportion to the amount of risk expressed in the probabilistic range of input values. If all risks are totally expressed in the probabilistic determination of the range in values of the inputs, a riskless discount rate must be used. This is in contrast to a risk-adjusted discount rate that may be used for nonprobabilistic evaluation methods, such as scenario analysis. A computer program using a method called Monte Carlo simulation can then be used to generate hundreds of variations of cash flows (i.e., hundreds of scenarios) and NPVs for an individual project. The statistical distribution of the NPVs is then analysed to determine the worth of the investment opportunity. The mean of the NPVs obtained by Monte Carlo simulation represents the statistically defined expected value of the project (Torries, 1998).

EXAMPLE; A LARGE COAL MINE

In this part, a large coal mine project is examined as a financial mining project. The life of project is 29 years. Basic data and the DCF are demonstrated in Table 1. For reach to final goal (risk analysis of project), this table must set up in a spreadsheet as Excel and the main parameters must be connected together. In this table, interest rate parameter is set up separately for the future aims.

INTRODUCING A POWERFUL SOFTWARE FOR RISK ANALYSIS

@Risk is a powerful software in risk analysis of projects that adds in Excel Spreadsheet and is capable of incorporation with Excel’s modelling abilities with best efficiency. This software is applied for the analysis of projects with probabilistic parameters. Probability distribution of each parameter must be inserted into for its cell. Then @Risk runs the model by the Monte Carlo and Latin Hypercube methods and random numbers generation for many times. Then, project’s parameters are analysed and finally, outputs and their probability distributions are presented. Examining the output data and with respect to expectations, the counted risk of project will be gained. Also, @Risk is capable of presenting output data graphically that cause of better realisation from the results. In this paper, we have used the newest edition of software (4.5 Industrial Version) that has more capabilities than older versions.

PROCESS

Definition of input and output parameters

Two parameters of interest rate and coal price are selected for input parameters for their oscillation and variation in two past decades based on historical data. These historical data are collected and their probability distributions are defined by curve fitting. The results of these definitions for interest rate and coal price are shown in figures 2 and 3 respectively. The results show that the interest rate follows Logistic distribution and also coal price follows the Exponential distribution.

Afterwards, the output cells must be defined in the spreadsheet. In this study, final Net Present Value of the project (NPV) and present values of each year defined as the output parameters.

Simulation

After definition of inputs and outputs, we can run the model and perform simulation. But, before this, settings of simulation must be set up. The model should be run with logic (sufficient) iterations in the way that convergence is yielded. Table 2 shows the output data resulted from simulation.

Also, Figures 4 and 5 shows the final probability distribution and histogram for the NPV output parameter. Analysing of these graphs, we can measure the counted risk of the project respect to expectations.
Table 1 - Basic data and DCF for the coal mine project

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Figure 2 - probability distribution for interest rate parameter

Figure 3 - probability distribution for coal price parameter

Figure 4 - Final probability distribution for NPV
In figure 5, we have demonstrated an applicable graph nominated Tornado for NPV parameter that be used for sensitivity analysis of parameters affected in the project. Analysing this graph, we realize that the most important input parameter that affects the NPV, is interest rate.

In figure 6, we have demonstrated an applicable graph nominated Tornado for NPV parameter that be used for sensitivity analysis of parameters affected in the project. Analysing this graph, we realize that the most important input parameter that affects the NPV, is interest rate.

CONCLUSIONS

Risk analysis has moved from the research laboratories of financial economists into the wider world. The techniques now available include a powerful array of simulation methodologies- Monte Carlo and Latin Hypercube sampling and natural language computation (Hacura et al. 2001). They allow the evaluator to provide much more objectives assessments of the risk of a mineral prospect. These methods are available in relatively inexpensive software capable of functioning in most modern personal computers. The most serious difficulty currently encountered in the stochastic risk analysis of mining projects is lack of historical information about the industry. In this study, we analysed the risk of a large coal mining project in some stages briefly. Surveying the output graphs, the manager will be able to measure counted risk of project.

Results including tables and graphs showed that this project has a low risk with taking about interest rate and price as probabilistic parameters in the analyses. However, sensitivity analysis show that the most important input parameter that affects the NPV, is interest rate and the managers must pay high attention in the project period.
REFERENCES


AN INFORMATION TECHNOLOGY KNOWLEDGE BASE FOR COAL MINING: WEBSITES AT THE UNIVERSITY OF WOLLONGONG

Naj Aziz¹, Richard Caladine¹, Wiebe Wilbers¹ and Lucia Tome¹

ABSTRACT: A new web portal “Coal Mine Science and Technology” has been established at the University of Wollongong to provide common access to a series of websites operating from its Information Technology servers. The portal links to three active websites and one that is in the process of development. The currently active websites are: Longwall Mining, Coal Mine Outbursts, and Bord and Pillar Mining websites. The fourth website on Heading Development is currently being prepared. All the active websites are designed with a common format and layout but differ in colour scheme. ACARP has funded the Coal Mine outbursts, and Heading Development websites while the other websites are partly funded by the University of Wollongong (UOW) and partly by the industry. The focus of the websites content has a technical orientation and provides the latest information and technology transfer aimed at the mining industry specifically for Australia and generally world wide. The content of all the websites has a predominantly Australian focus.

INTRODUCTION

The initial creation/establishment and subsequent success of the Longwall Mining website, has demonstrated that the UOW is the leading provider of an information technology knowledge base in coal mining. The second in the series of websites to follow was the ACARP sponsored website on Coal Mine Outbursts. This is now followed by the launching of a third website on Bord and Pillar Mining. All these three websites are currently functioning and have attracted significant attention from both the industry and education institutions world wide, both in mining and related disciplines.

A common web portal has been established to act as an umbrella of single access, or gateway to all UOW coal mining related websites. Known as Coal Mining Science and Technology Website (CMST), this common portal enables better organisation of the current (UOW) mining websites and provides unification of different information knowledge features as shown in Figure 1. Recently a new space has been created to accommodate the currently under construction and yet to be launched website on Heading Development. The website is aimed at a variety of issues related to improvements in the speedy development of headings to serve high production longwall faces.

At present the content of all three functioning websites is focused on the Australian coal mining operations. However, plans are underway to include information on mining from overseas operations and related research that will benefit the Australian coal industry. Already the Coal Mine Outbursts website contains reports from overseas practices and research, notably from Poland and links are already in place with NIOSH of the United States of America.

Presently the Coal Mining Science and Technology Web Portal has three functioning websites. The Heading Development website is yet to be realized as the project is in the infancy stage. The current strategy is that all the websites will be uniform in layout and structure, but differ in colour. The common look to the websites is intended to:

- remind the user (browser can be confused with a web browser) of the location and source of the websites, in this case the University of Wollongong,
- facilitate the easy navigation of each website, through familiarisation and consistency, and
- make available common menu items for all three websites such as glossary, the links menu (to a certain extent) and the contact details of the development team.

The web address or Uniform Resource Locator (URL) of the Coal Mining Science and Technology portal is: http://research.uow.edu.au/coal

¹ University of Wollongong, Wollongong
WEBSITE CONSTRUCTION METHODS

All three functioning websites, namely: Longwall Mining, Coal Mine Outbursts, and Bord and Pillar Mining websites were constructed with several tools. In the first instance the sites were constructed using a text-based HTML (Hyper Text Mark-up Language) editor. Graphics and photographs for all sites were created in Photoshop and loaded to the site in the JPEG format. Adobe Dreamweaver and a small amount of JavaScript were incorporated to each site. Dreamweaver, while more expensive than text-based HTML editors, has the advantage of providing two windows: one in which the developing site is visible and another for the code. The JavaScript has been incorporated to assist in navigation of the site as it provides the interactive menus.

All three mining websites has a similar look, with lateral navigation menu bars beneath the website banner. The menu bar contains a range of menus with drop down lists of links to different topics. Sub-menus are incorporated in some menus which also link to the core content of the website.

CONTENTS OF THE WEBSITES

The three functioning websites linked from the Coal Mining Science and Technology portal are similar in design but differ in page colour and content.

Longwall Mining website: http://www.uow.edu.au/eng/longwall

The Longwall Mining website was the first website to be constructed and is also the most frequently visited within the portal. The technical contents of the Longwall website are contained in seven menus and sub-menus with drop-down lists. These menus and their details are summarised as follows:

- **Overview:** About the site, history and methods, glossary, Australian longwalls.
- **Equipment:** shearer, plough, powered supports, AFC, pantechnicon, beam stage loader, communications and environmental controls.
- **Ground Control:** Abutment pressures, support capacity, gate road support, chain pillar design, ground subsidence, instrumentation, difficult conditions.
- **Ventilation:** Ventilation systems, dust control, auxiliary ventilation, gas drainage, spontaneous combustion.
- **Face Transfer:** Bolt up cycle, equipment recovery, Heusker system (DVD).
- **Heading development.**
Punch Longwall.

Links: International longwall news, NIOSH, Department of Mineral Resources, and Coal services NSW, Joy Mining Machinery, and DBT Mining.

Generally the Longwall Mining website has maintained its progressive edge on different aspects of longwall mining. The contents are regularly updated to maintain currency of technical information and of the technologies used. Issues that require future attention include increasing the numbers of references and publications as well as the inclusion of some key papers or articles which have had significant impact to the development of longwall mining technology.

Coal Mine Outbursts Website: http://www.uow.edu.au/eng/outburst/

The technical content of the Coal Mine Outbursts website is specifically related to mine gas and outburst control. Topics contained in the six menus of the navigation bar and sub-menus are as follows:

- **Overview**: Site map, definitions, aims and objectives of the website, contacts, glossary, and picture gallery.
- **Factors**: Including geological conditions, and coal properties.
  - Geological conditions including: Depth of mining, faults and folds, seam thickness, gas environment, gas content and mining induced stresses.
  - Coal properties including: coal seam strength, coal rank, coal permeability, volumetric change and cleat and joints.
- **Management**, including:
  - Prediction: Geology, prediction indices, monitoring, geophysical, gas environment, and gas content.
  - Prevention: Ventilation, gas threshold value, gas drainage, and borehole survey techniques.
  - Control: Ground de-stressing, gas drainage, borehole survey technique, hydro-Facing, pulse infusion shotfiring, outburst hazard control, and outburst management plan.
- **Research and Development**, including:
  - ACARP and NERDDP reports.
  - Seminar presentations, in power point slides, held in both Mackay and Wollongong.
  - Publications, both from Australia and international.
  - Reference list of past publications, totalling more than 300, and
  - John Hanes’ library of quality photographs and drawings from several Australian outburst sites spanning up to 30 years.

ACARP reports are “End of Project” reports. A number of relevant and important reports are currently available on the outburst website. They include;

- Coal Mine Outburst Mechanism, Thresholds and Prediction Techniques, by Ian Gray.
- ACARP Project C14032.
- Outburst Scoping Study - John Hanes.
- ACARP project, C 4034), Out bursting Scoping Study - March 1996 (Lama & Bodziony).

The website includes seminar presentations from both Wollongong and MacKay outburst seminars, held twice a year in each location. The PowerPoint presentation list starts from 2002 with a single paper being identified and uploaded, and in 2007 there were eight presentations in each of MacKay and Wollongong seminars.

Publications include full text papers, both Australian and international papers published since 1980. The Australian papers include presentations mostly held in various Aus IMM Conferences and seminars including (Aziz et al, 2007):

ii) 1981 Ignitions fires and explosions.
iii) 1982, Seam gas drainage with particular reference to the working seam, Wollongong.


vi) 1991 - 11th International Conference on Ground control in mining, University of Wollongong/ Illawarra Branch, Wollongong.

vii) 1992 Symposium on Coalbed Methane - Research and Development in Australia, November, Townsville, Australia.


ix) COAL2002- 3rd Coal Operators Conference, Coal 2002, Wollongong 


xii) COAL - 2005, 6th Underground Coal Operators’ Conference, 26 – 28 April, Brisbane. 


At the time of writing more than 250 references are listed in this section, mainly dealing with journal, conference, and other publications spanning more than 60 years. This website is constantly revised and updated. Figure 2 shows a web page typical of the outburst reference section.

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**Figure 2 - Typical reference list page of the Coal Mine Outbursts website**

*Links:* links are provided to selected websites that provide specific additional information. The selected websites are various recognised entities, technical websites, such as, ACARP, CSIRO, NIOSH (USA), Lunagas, and specialist companies such as, Sigra Pty Ltd, Luna gas, and Valley Longwall, Joy Mining.

*Discussion:* this section is aimed to establish chat room for direct dialogue, but is not yet fully operational.


The technical content of the website is specifically focused on Australian methods of Bord (room) and Pillar mining. Topics contained in the six menus in the navigation bar and sub-menus are as follows:

- **Overview:** Includes information about the website, coal deposits and types, history of coal mining in Australia and glossary of technical terms.
• **Methods**: Methods of bord and pillar mining including the first working in a tabular deposit, traditional methods of pillar extraction, panel and pillar extraction, and modern pillar extraction methods.

• **Equipment**: An overview of the equipment used including continuous miners, shuttle cars and continuous haulage systems, mobile breaker line support systems, belt conveyors, and ancillary equipment.

• **Ground Control**: Contains pillar design and principles of ground support.

• **Ventilation**: General introduction to bord and pillar ventilation systems. Other topics include dust control, auxiliary ventilation, and soon to be added spontaneous combustion.

• **Links**: International longwall news, NIOSH, Department of Mineral Resources, and Coal services NSW, Joy Mining Machinery, and DBT Mining and others.

Figure 3 shows a typical page from the Bord and Pillar Mining website.

**Figure 3 - Bord and pillar mining website**

**WEBSITE SURVEY AND STATISTICS**

Recently a new statistical counter was introduced on each of the three websites. There have been 32,139 hits on the Longwall website, while the Outburst website received 4,900 hits and the Border and Pillar Mining website, launched in October 2007, has had approximately 2500 hits. The higher number of visitors to the Longwall website is expected due to its long-term establishment (launched 1999) as well as being the most widely publicised website.

Figures 4 to 6 show the statistics on the number of visits since August 2007. As can be seen the websites have both national and international appeal. The trend of the access varies for different periods of the year and from one country to another. Clearly, visitors from Australia and North America are the two most regular users. It is envisaged that the interest from Australia will grow even further once the website is formally launched with wider publicity and presentations in conferences and seminar.
Figure 4 – Longwall Mining Website monthly visits statistics (August - December 2007)

Table 2 - Countries that visited the Longwall Mining Website (August - December 2007)

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<td>48</td>
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<td>8.62%</td>
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<td>27</td>
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<td>United States</td>
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<td>15</td>
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<td>11</td>
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Figure 5 - Coal Mine Outbursts Website monthly visits statistics (the next page) (August – December 2007)
Table 4 – Countries visiting Coal Mine Outbursts Website (August - December 2007)

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<td>Japan</td>
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Figure 6 - Bord and Pillar Website monthly visits statistics (August - December 2007)
Table 5 - Countries that visited the Bord and Pillar Mining Website (August - December 2007)

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CONCLUSIONS

An online information management system consisting of a portal connecting websites on coal mine science and technology has been developed to provide the coal mining industry and educational institutions with the necessary information and knowledge on Longwall Mining, Coal Mine Outbursts and Bord and Pillar Mining. A fourth website on Heading Development is currently being developed.

The portal: Coal Mining Science and Technology (http://research.uow.edu.au/coal) provides ready access to these four websites, allows for common information to be readily accessible and has the capacity for further development. The portal and websites have met the required objectives of providing easy access to the experience, knowledge and information acquired by the coal mining industry, research organisations and universities in a quality-controlled environment.

REFERENCES


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