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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 Vrahas 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 Convener 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, Ellton 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[1], Geoff Lyman[1] and Alan Cocker[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.

[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.

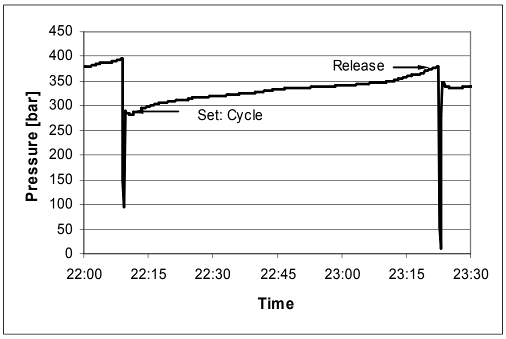


Figure 1 - Pressure *versus* time trend for a typical loading cycle where no yield occurs, indicative of a support of adequate capacity and appropriate set pressure.

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.

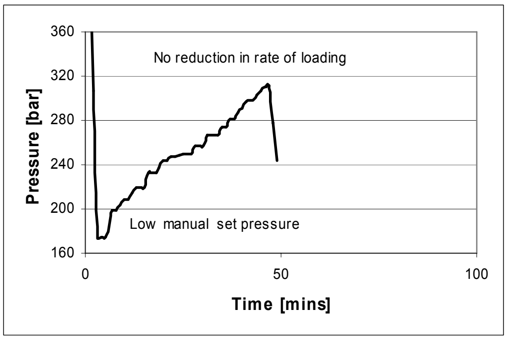


Figure 2 - Typical loading pattern with set pressure too low for the conditions

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. With shields rated at about 100 tonnes per square meter before the shearer takes the next cut, no problems have been observed by the authors of this paper at set pressures of 60% or greater. This equates to an initial support density of 60 tonnes per square meter before the cut. With the same shield rating significant problems have been observed at set pressures below 40% of yield, which equates to 40 tonnes per square meter before the cut. Based upon these observations the authors of this paper would recommend a set pressure equating to a minimum support density of 40 tonnes per square meter before the cut and would consider 60 tonnes per square meter to be better if such set pressures can be practically achieved. If such set pressures are not attainable in the conditions then mining as rapidly as possible will mitigate subsequent roof control problems.

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.

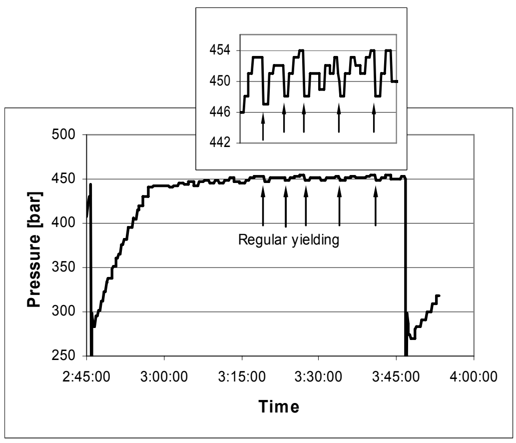


Figure 3 - Support loading for yielding without stabilisation indicating a support with inadequate capacity.

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.

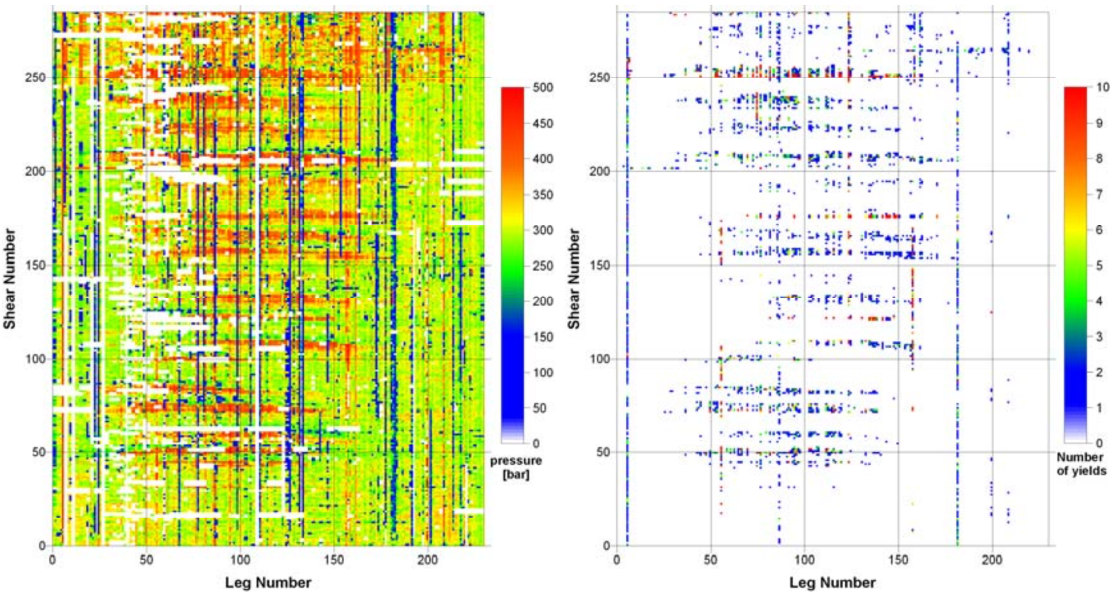


Figure 4 - TWAP (left) and number of yield events for July 10th- August 7th inclusive.

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.

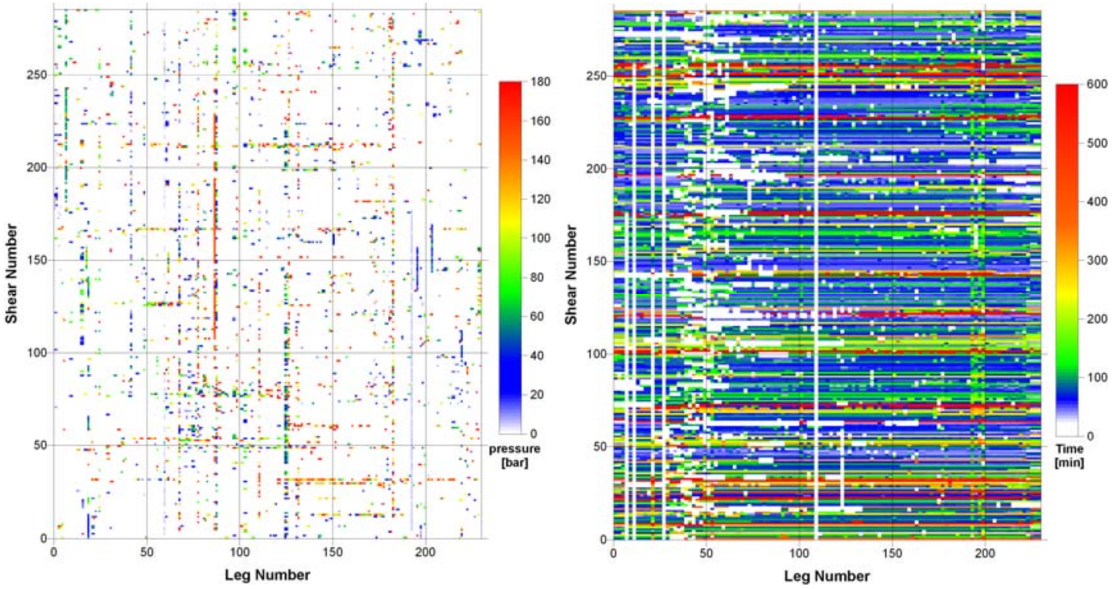


Figure 5 - Supports having a set pressure below 180 bar (left) and cycle time (set to release) for the supports (right).

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.

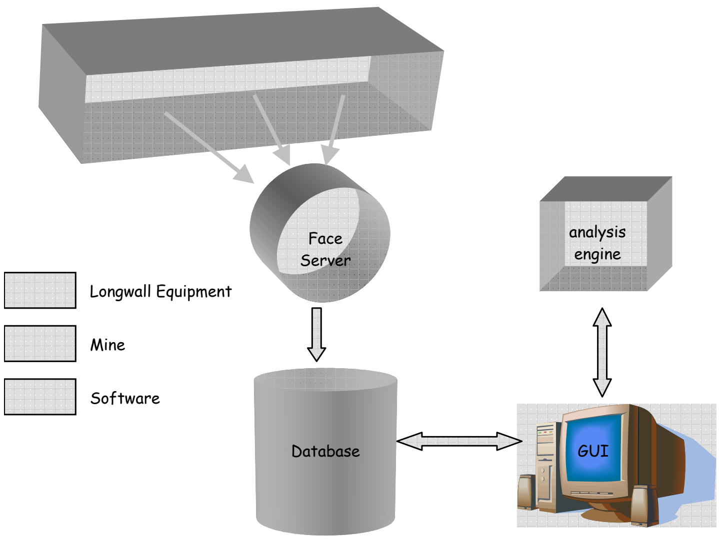


Figure 6 - On-line software schematic

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.

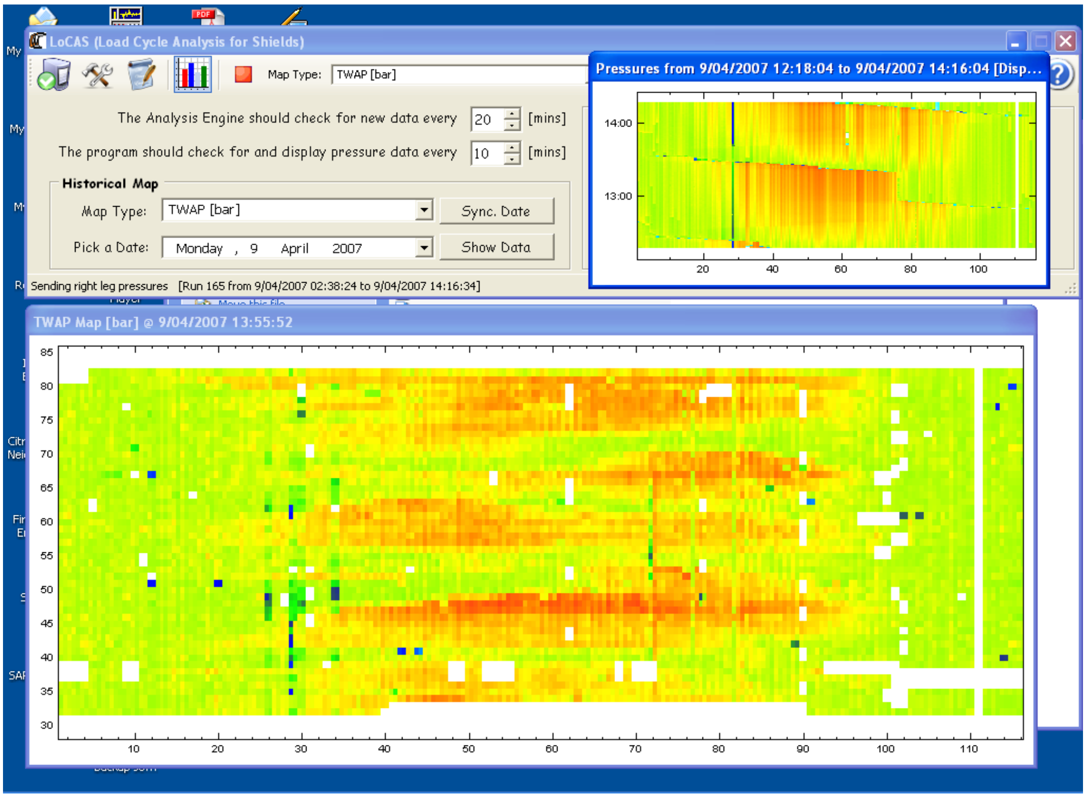


Figure 7 - Control panel, in this case showing TWAP on the main screen and instantaneous pressures top right

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.

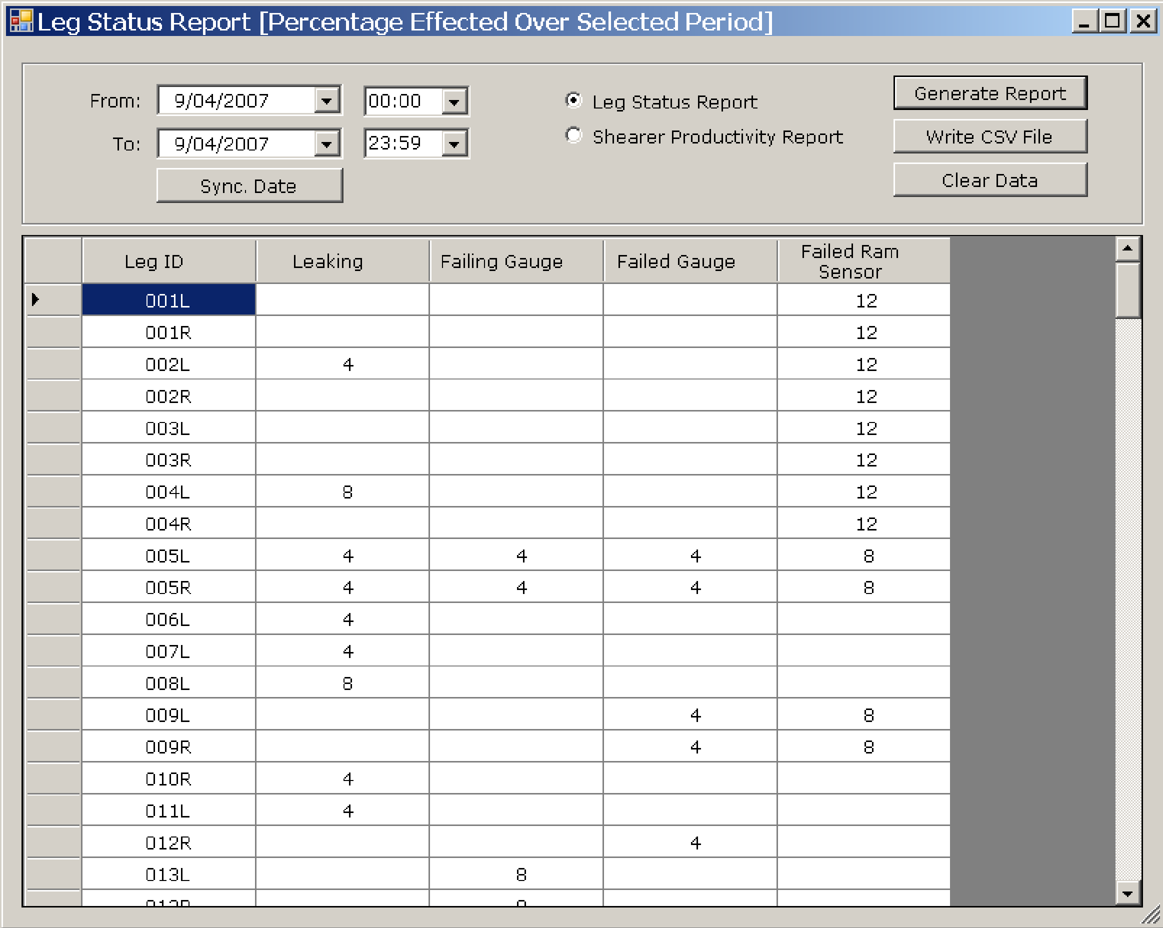
A report is generated of the percentage of time during a user defined period that identified problems (leaking leg, failing gauge, failed gauge, failed ram sensor) occurred; Figure 8.

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.

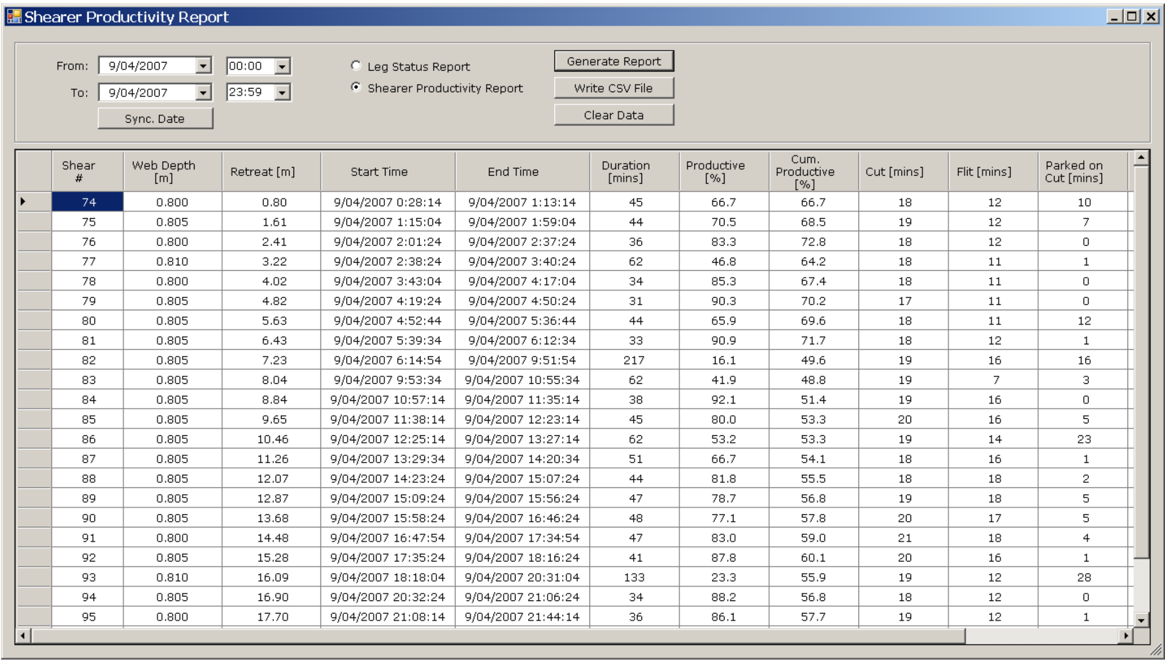
CONCLUSIONS

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



Leg ID	Leaking	Failing Gauge	Failed Gauge	Failed Ram Sensor
001L				12
001R				12
002L	4			12
002R				12
003L				12
003R				12
004L	8			12
004R				12
005L	4	4	4	8
005R	4	4	4	8
006L	4			
007L	4			
008L	8			
009L			4	8
009R			4	8
010R	4			
011L	4			
012R			4	
013L		8		

Figure 8 - Report on potential faults on individual legs



Shear #	Web Depth [m]	Retreat [m]	Start Time	End Time	Duration [mins]	Productive [%]	Cum. Productive [%]	Cut [mins]	Flit [mins]	Parked on Cut [mins]
74	0.800	0.80	9/04/2007 0:28:14	9/04/2007 1:13:14	45	66.7	66.7	18	12	10
75	0.805	1.61	9/04/2007 1:15:04	9/04/2007 1:59:04	44	70.5	68.5	19	12	7
76	0.800	2.41	9/04/2007 2:01:24	9/04/2007 2:37:24	36	83.3	72.8	18	12	0
77	0.810	3.22	9/04/2007 2:38:24	9/04/2007 3:40:24	62	46.8	64.2	18	11	1
78	0.800	4.02	9/04/2007 3:43:04	9/04/2007 4:17:04	34	85.3	67.4	18	11	0
79	0.805	4.82	9/04/2007 4:19:24	9/04/2007 4:50:24	31	90.3	70.2	17	11	0
80	0.805	5.63	9/04/2007 4:52:44	9/04/2007 5:36:44	44	65.9	69.6	18	11	12
81	0.805	6.43	9/04/2007 5:39:34	9/04/2007 6:12:34	33	90.9	71.7	18	12	1
82	0.805	7.23	9/04/2007 6:14:54	9/04/2007 9:51:54	217	16.1	49.6	19	16	16
83	0.805	8.04	9/04/2007 9:53:34	9/04/2007 10:55:34	62	41.9	48.8	19	7	3
84	0.805	8.84	9/04/2007 10:57:14	9/04/2007 11:35:14	38	92.1	51.4	19	16	0
85	0.805	9.65	9/04/2007 11:38:14	9/04/2007 12:23:14	45	80.0	53.3	20	16	5
86	0.805	10.46	9/04/2007 12:25:14	9/04/2007 13:27:14	62	53.2	53.3	19	14	23
87	0.805	11.26	9/04/2007 13:29:34	9/04/2007 14:20:34	51	66.7	54.1	18	16	1
88	0.805	12.07	9/04/2007 14:23:24	9/04/2007 15:07:24	44	81.8	55.5	18	18	2
89	0.805	12.87	9/04/2007 15:09:24	9/04/2007 15:56:24	47	78.7	56.8	19	18	5
90	0.805	13.68	9/04/2007 15:58:24	9/04/2007 16:46:24	48	77.1	57.8	20	17	5
91	0.800	14.48	9/04/2007 16:47:54	9/04/2007 17:34:54	47	83.0	59.0	21	18	4
92	0.805	15.28	9/04/2007 17:35:24	9/04/2007 18:16:24	41	87.8	60.1	20	16	1
93	0.810	16.09	9/04/2007 18:18:04	9/04/2007 20:31:04	133	23.3	55.9	19	12	28
94	0.805	16.90	9/04/2007 20:32:24	9/04/2007 21:06:24	34	88.2	56.8	18	12	0
95	0.800	17.70	9/04/2007 21:08:14	9/04/2007 21:44:14	36	86.1	57.7	19	12	1

Figure 9 - Screen dump of shearer production report

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.

ACKNOWLEDGEMENTS

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

Dan A Payne[1]

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).

[1] *BHP Billiton Pty Ltd*

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.

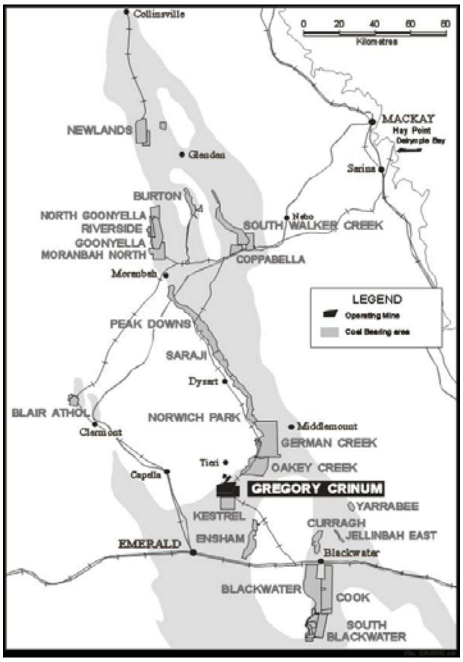

Figure 1 - Location plan of the Crinum Mine

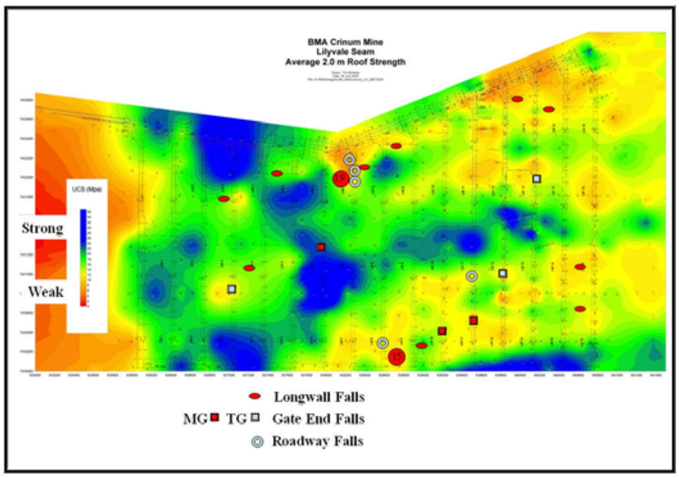

Figure 2 - 2m UCS contour overlay onto the Crinum mine plan with the locations of roof falls also plotted

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

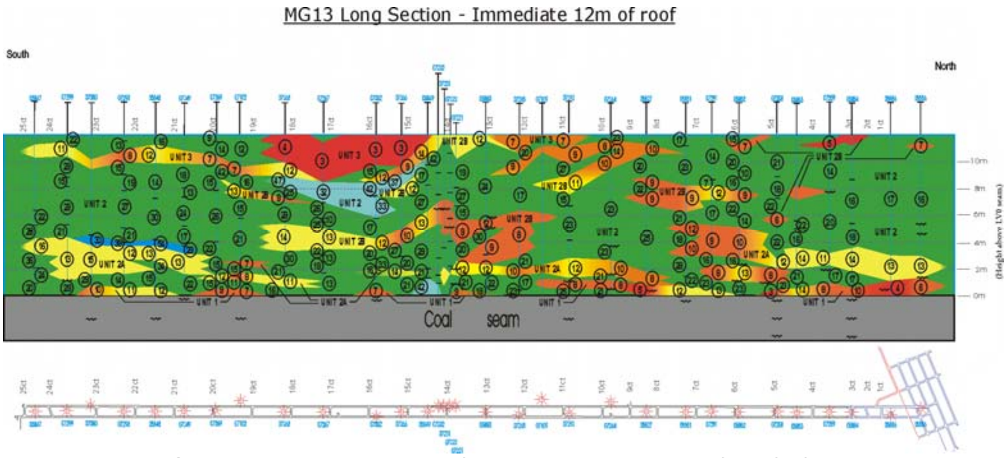

Figure 3 - Geomechanical section of the immediate 12 m of roof of Longwall 13 showing the 130 m borehole spacing

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.

Figure 4 - Initial section of the mine with straps and remainder of mine development with full roof mesh

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.

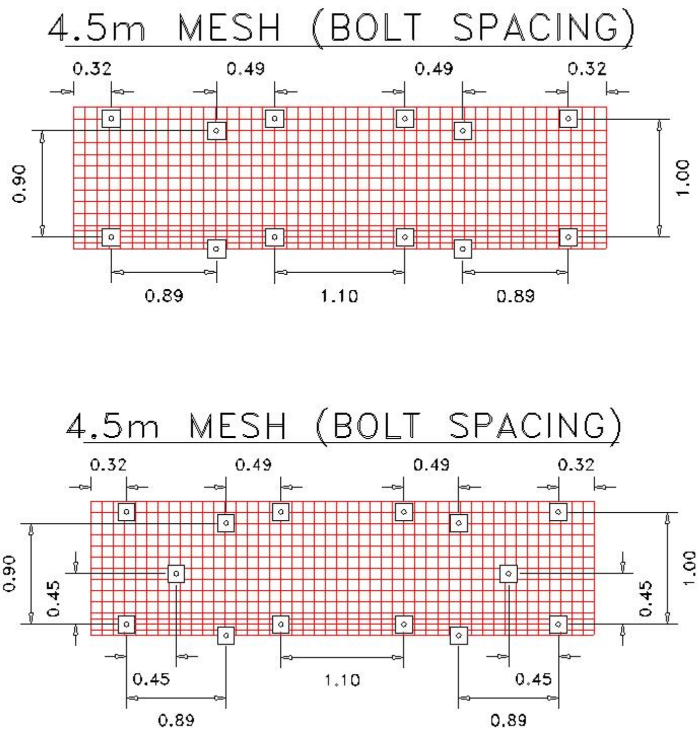

Figure 5 - 6 bolts per metre and 6:2 bolts per metre patterns

Figure 6 - Inbye lip of roof fall stopping at the 6:2 pattern showing line of roof failure.

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).

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

Figure 10 - Improvement in rib conditions and tailgate travel road conditions after changing from no mesh on pillars (left) to full mesh (right).

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).

Figure 11 - Tailgate blockside rib conditions with 300-500 mm unencapsulated cutable rib bolts (left). Blockside rib support with full encapsulation on development (right).

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 is 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.

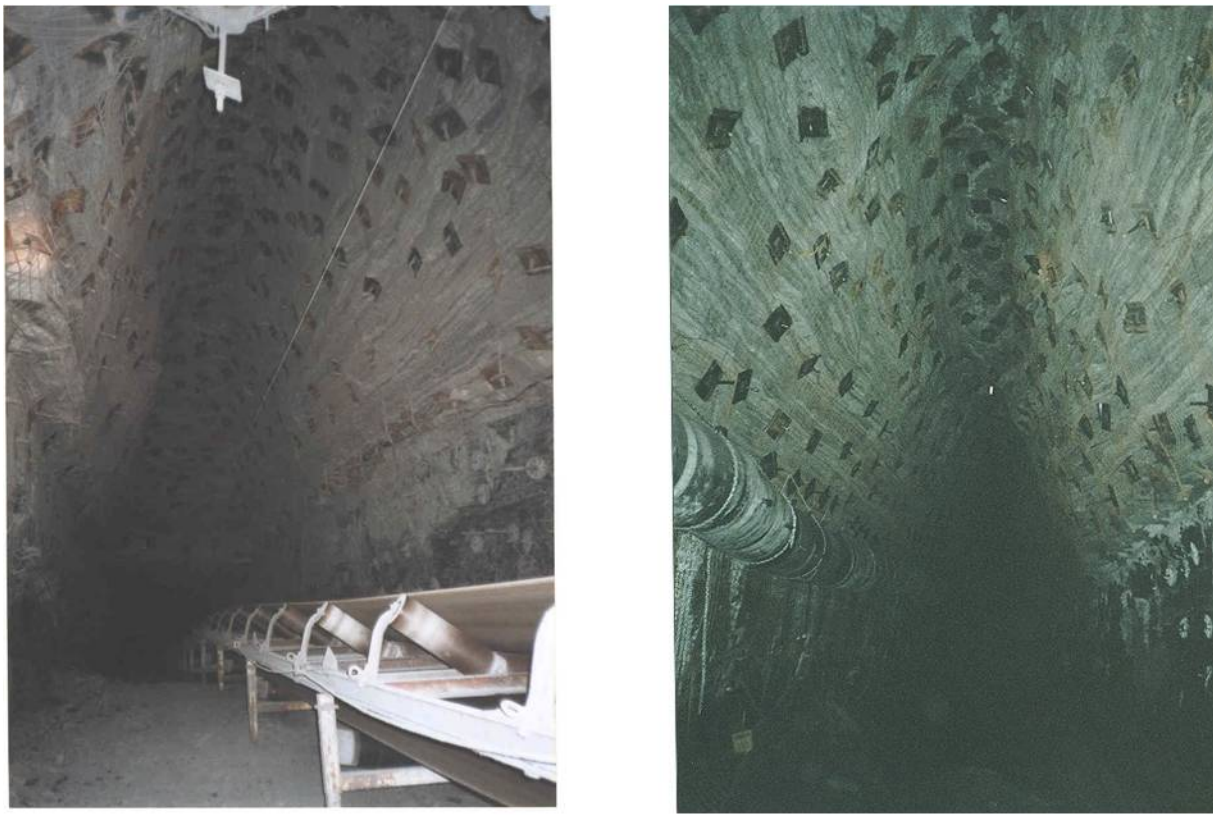

Figure 12 - Standard triangular shape of roadway roof fall cavities at Crinum

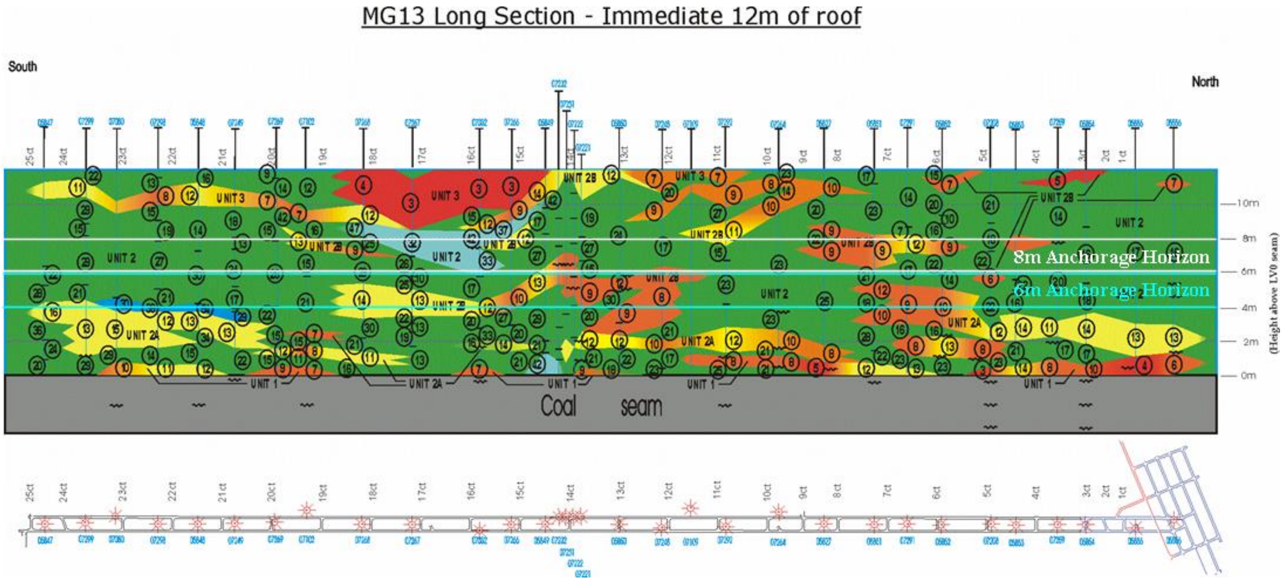

Figure 13 - Selecting cable bolt lengths based on effective anchorage horizon

Figure 14 - Maingate falls on the BSL in roof with post grouted cables

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.

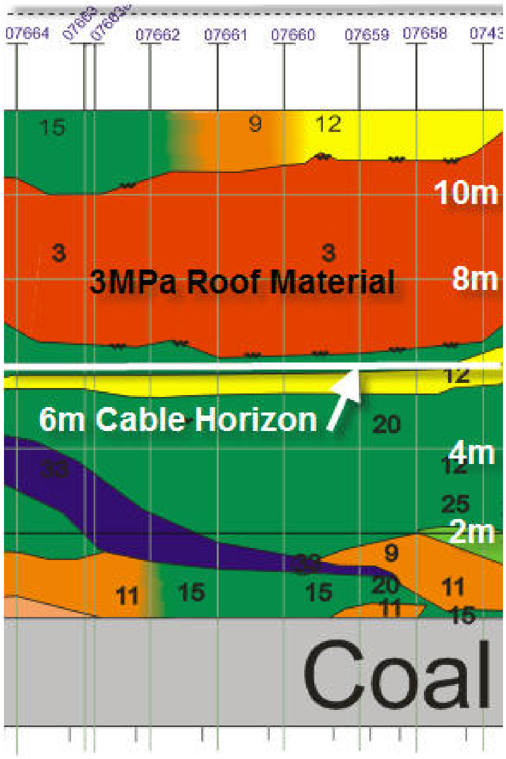

Figure 15 - Geomechanical roof section showing 6 m cable bolt horizon for LW16 installation road

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 flitted 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.

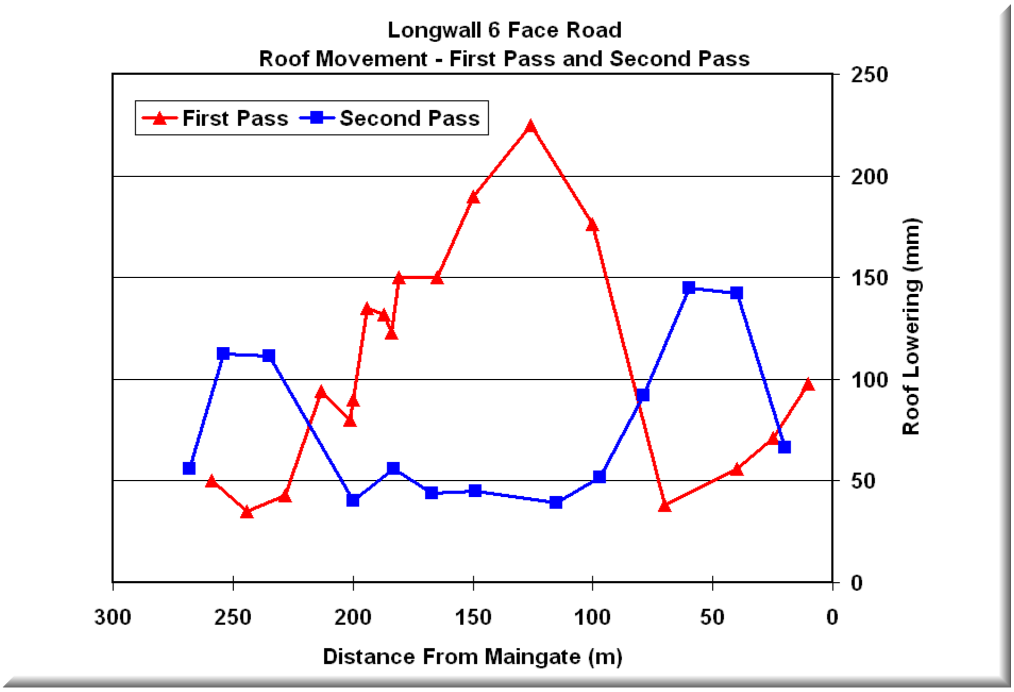

Figure16 - Roof movement first pass and second pass on the longwall 6 installation road demonstrating the reduction in roof lowering in the midface area where the most microfine cement was pumped into the roof.

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 spile 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 spile 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 2 0 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).

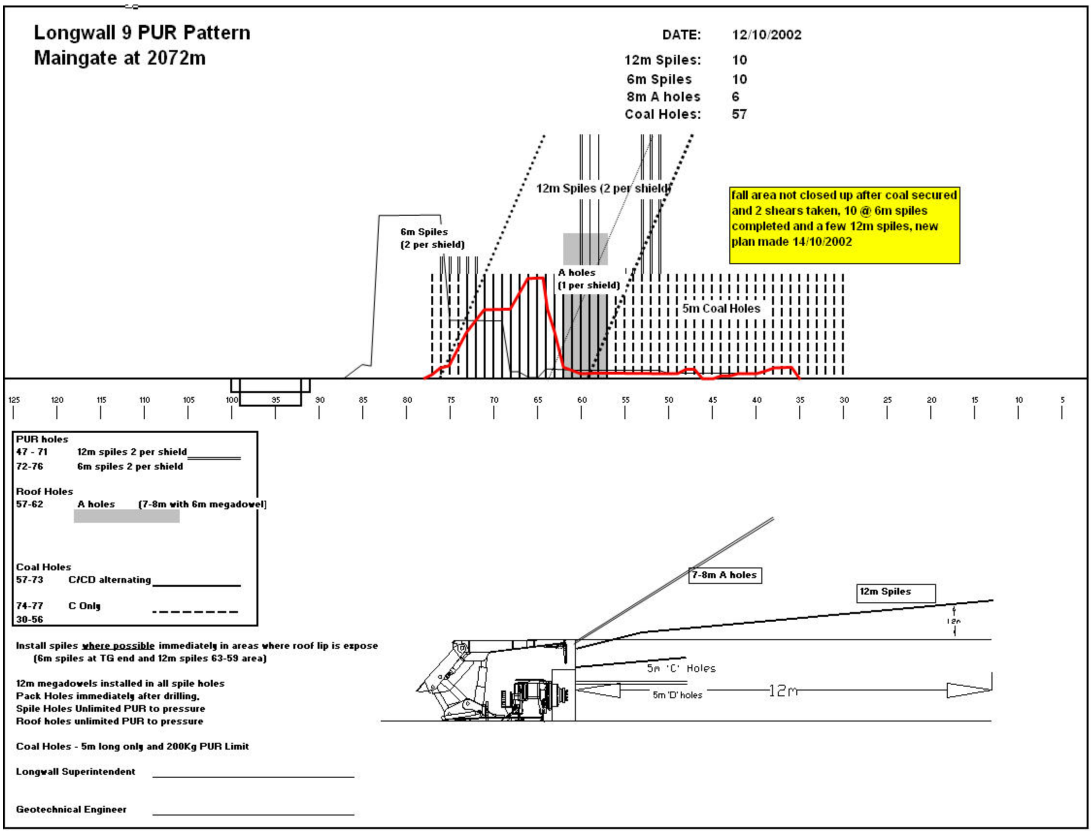

Figure 17 – PUR injection – longwall recovery support plan

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.

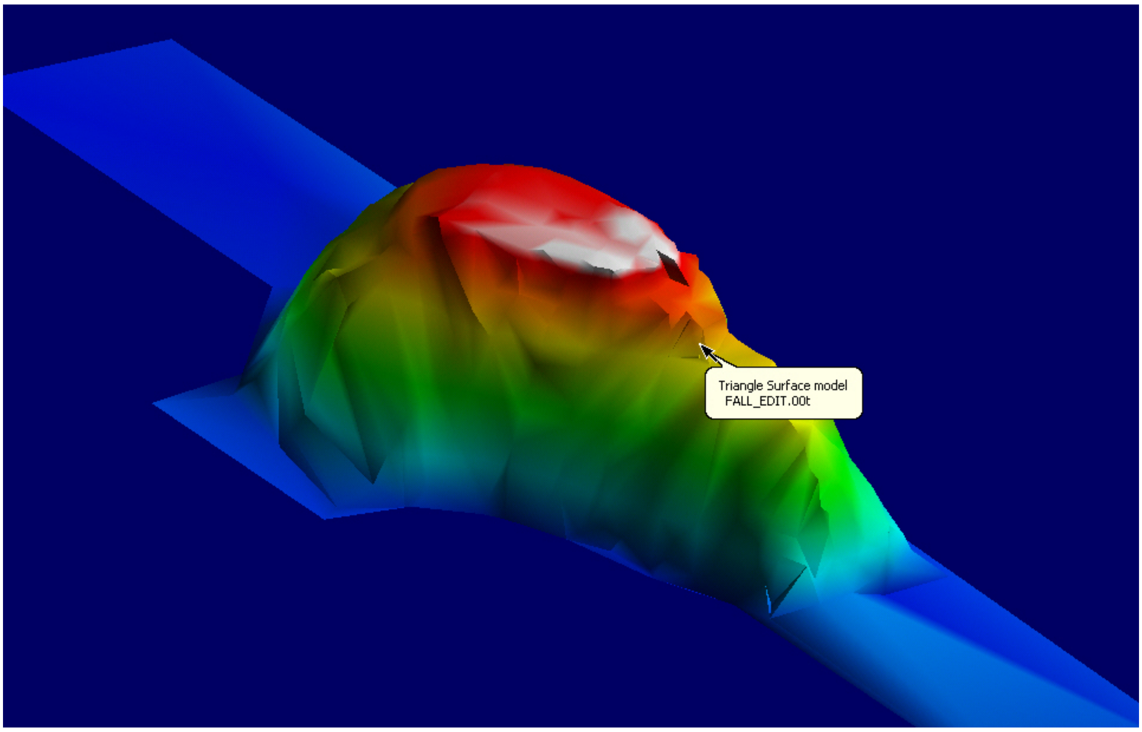

Figure 20 - 3D plot of roof fall cavity measurement using survey instrumentation

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

Crinum gateroad 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, Crinum 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 Crinum 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 telltales at every standard intersection and four point electronic extensometers at critical areas such as installation roads and takeoff roadway intersections. The electronic telltales 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 telltales and resistive potentiometer extensometers would be read by ERZ controllers and the data entered by the control room operator on shift. This included outbye telltales 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 telltales by converting to the clock it style tell tales. 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 telltales. 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.

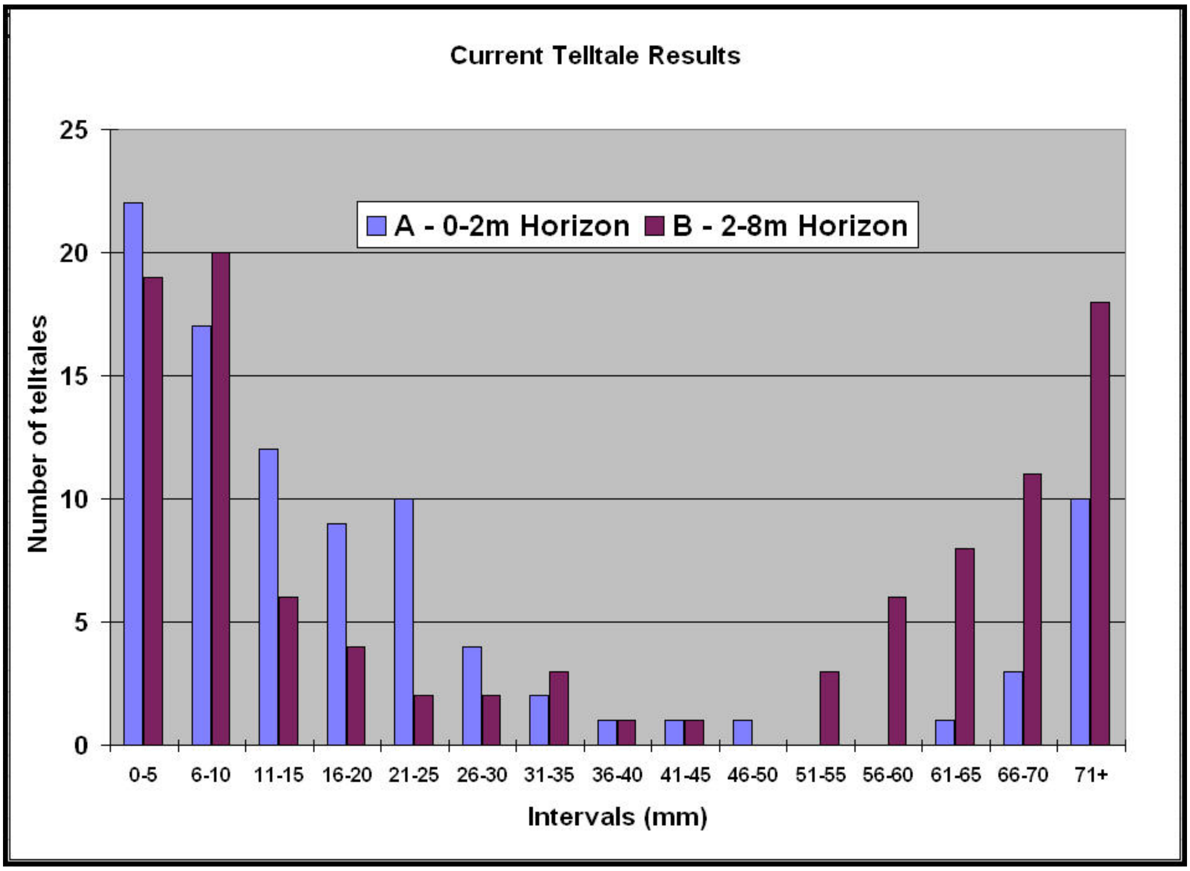

Figure 21 - Mine wide telltale results used to validate alarm levels.

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.

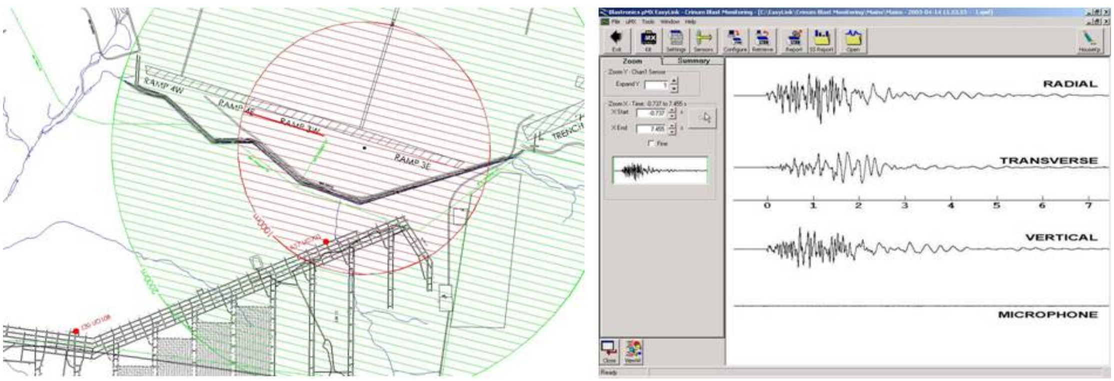

Figure 22 - Radial distance from open cut blast to underground workings (left). Blast vibration monitoring results from underground workings (right).

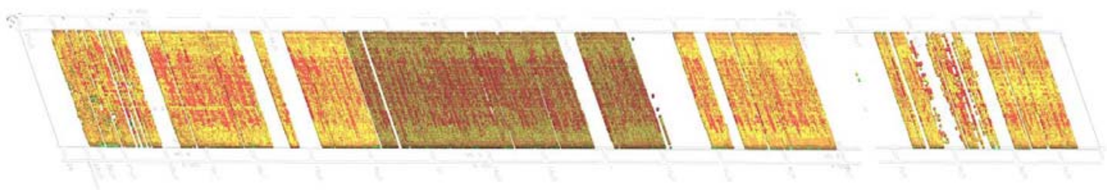

Figure 23 - Longwall shield monitoring results showing maximum leg pressure per cycle and a slight but not significant weighting cycle of 10 m

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 slabbing 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 scours 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^o 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 180^0 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 90^0 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.

Figure 24 - Typical face spall after limited downtime on the longwall (left). Inability of coal deflectors to secure the coal face (right).

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).

Figure 25 - Tailgate standing support reduced from 915 mm diameter 3 m centres to 700 mm diameter 4 m centres

Figure 26 - Improvement takeoff faceline support from polymer grid and resin bolts to flexible sray on membrane and split sets.

WATER INFLOW

Mining under Aquifers

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.

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.

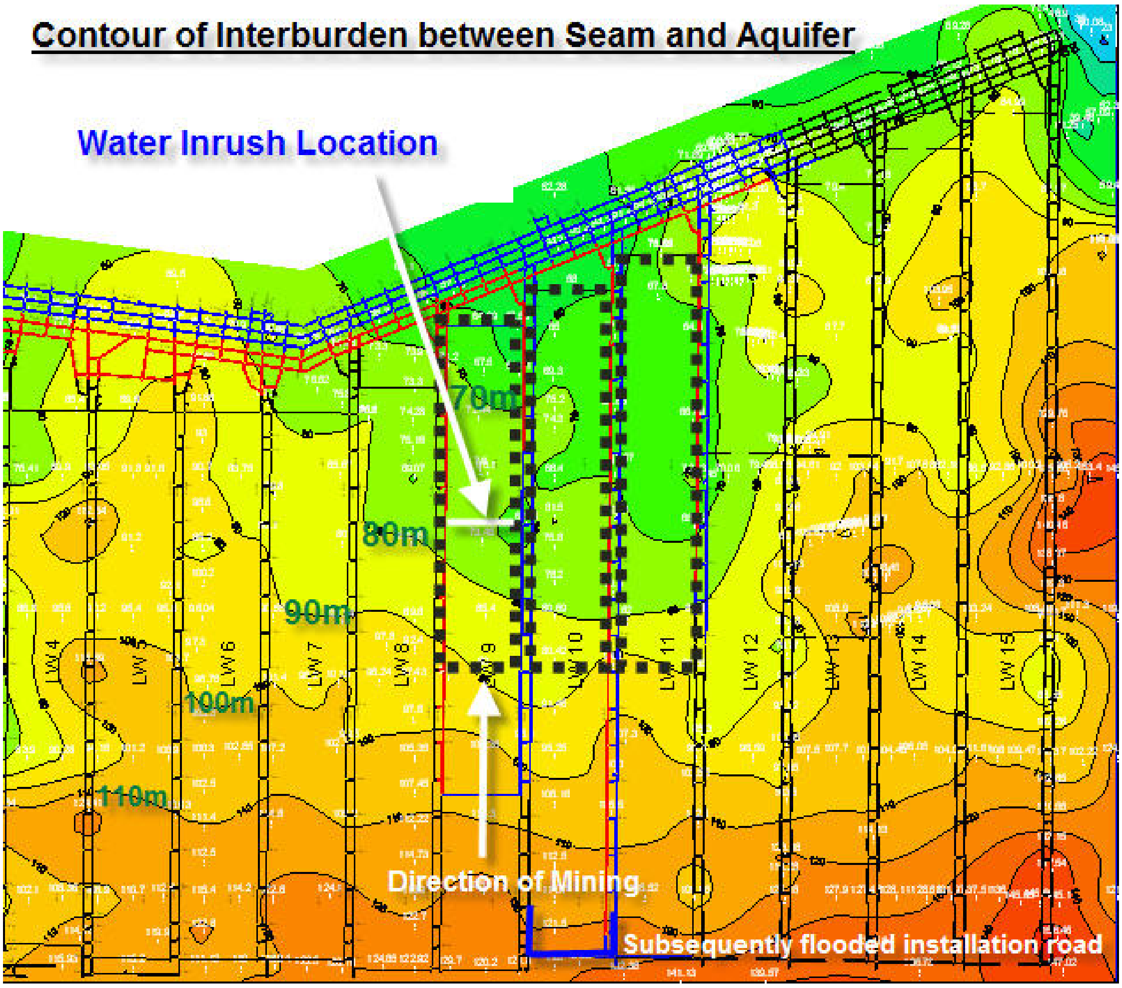

Figure 27 - Interburden isopach showing thickness of Permian strata between the seam and a tertiary aquifer

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

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

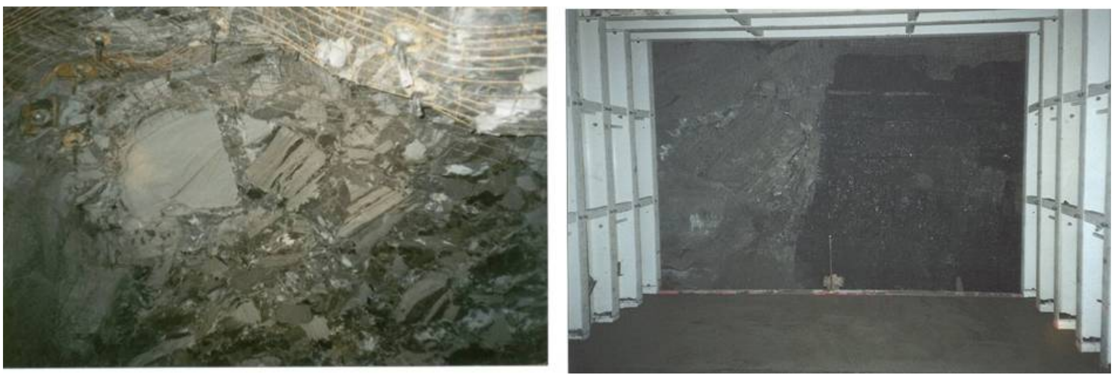

Figure 31 - Broken semiconsolidated material of the diatreme (left) and its contact boundary with the coal seam (right).

EFFECT OF GROUTING ON LONGWALL MINING THROUGH FAULTS

Terry Medhurst[1], Michael Bartlett[2] and Renate Sliwa[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).

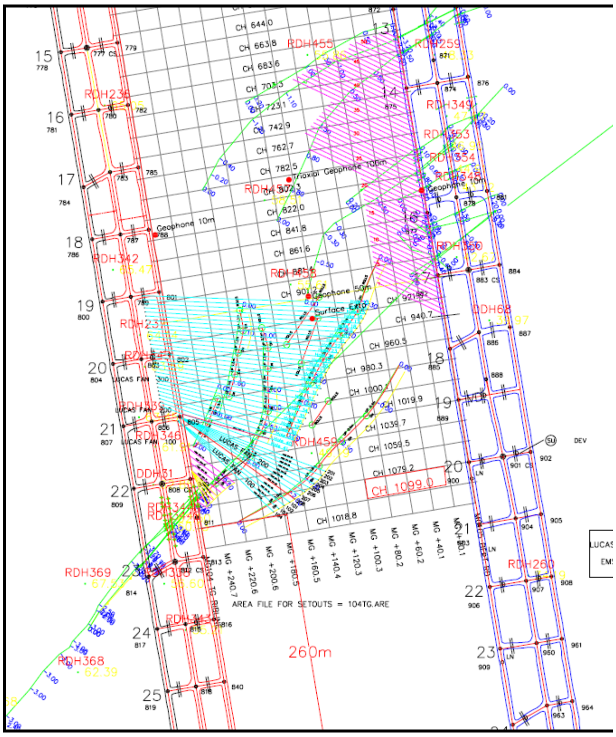

Figure 1 - Typical drill hole layout into LW block

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.

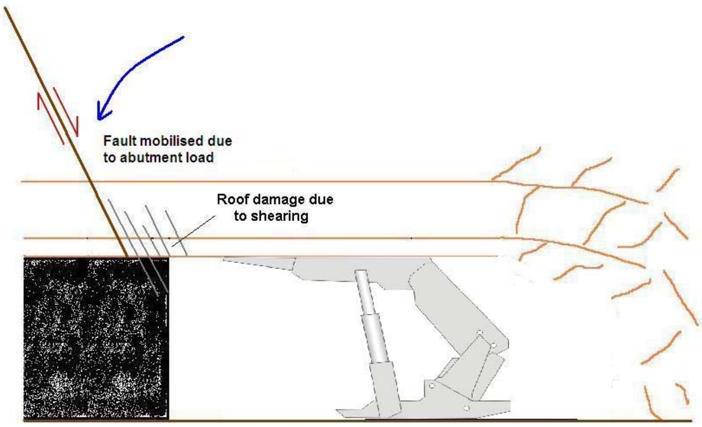

Figure 2 - Potential for roof damage due to vertical abutment loading

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

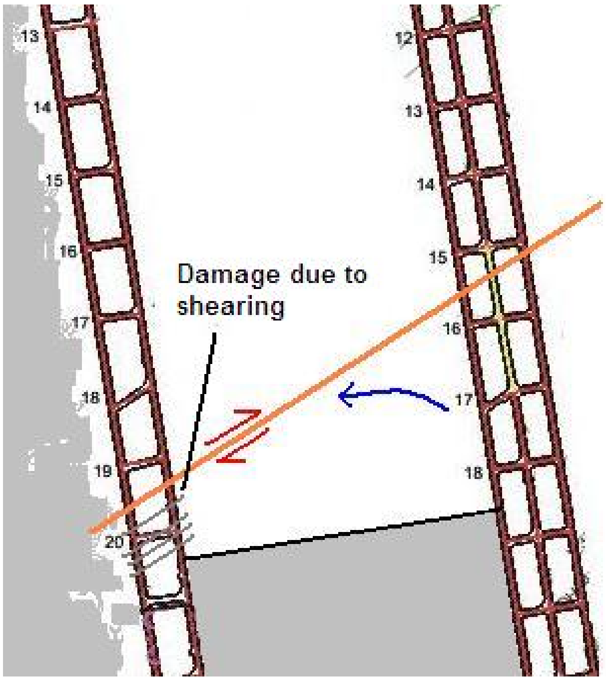

Figure 3 - Potential for damage due to horizontal abutment loading

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.

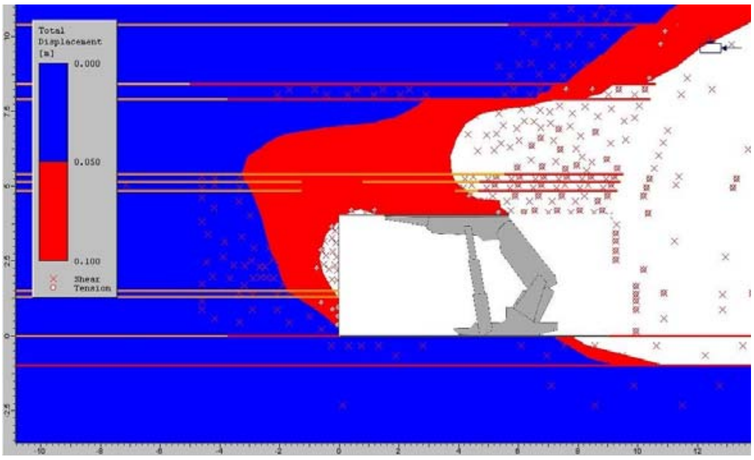

Figure 4 - Typical strata damage profile under 'soft' conditions

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.

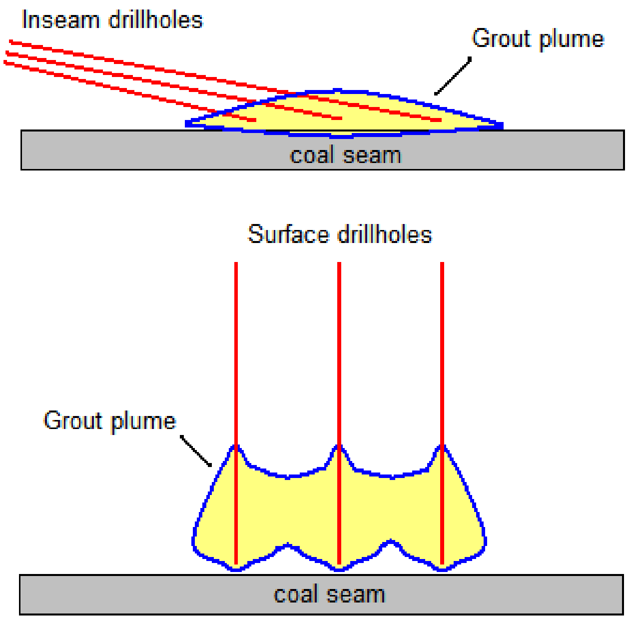

Figure 5 - Typical grout coverage from surface and inseam holes

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.

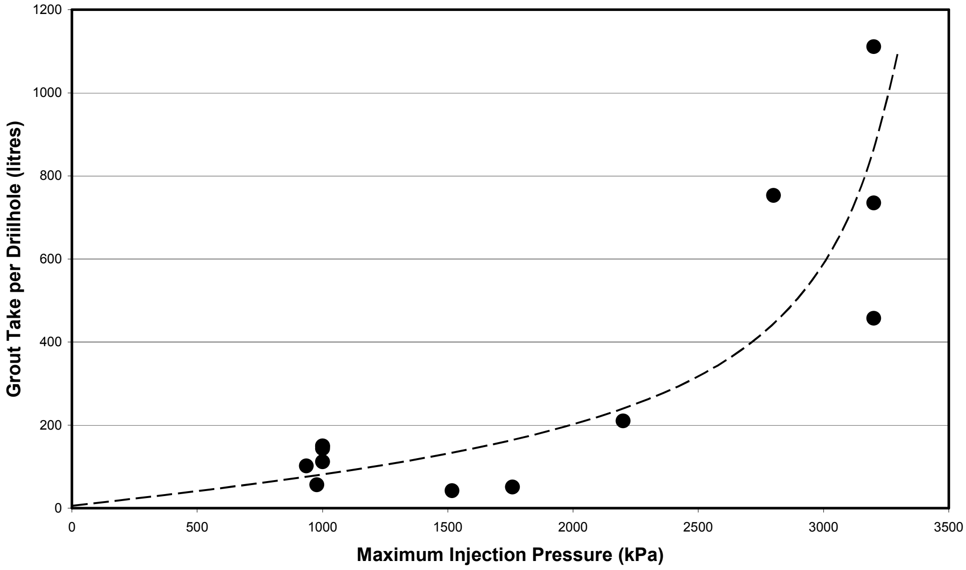

Figure 6 - Relationship between grout pressure and grout take

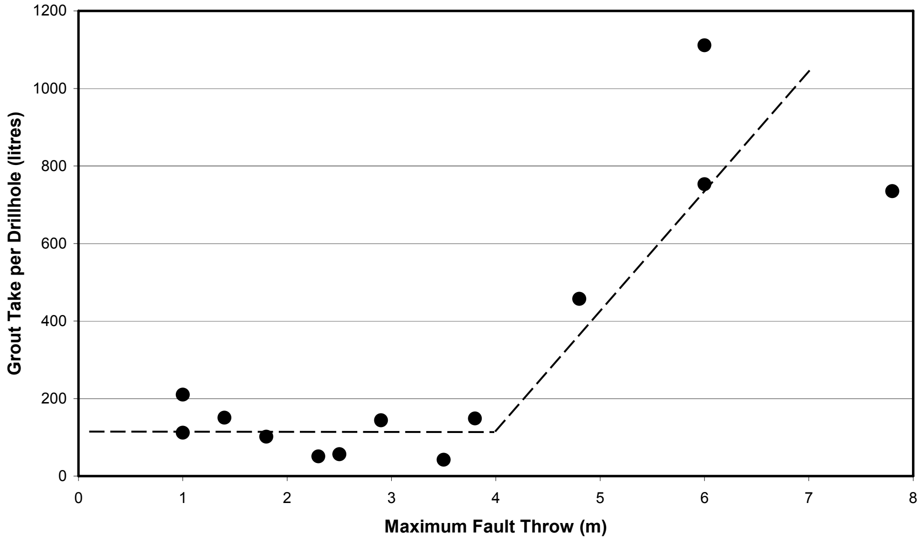

Figure 7 - Comparison of grout take and maximum fault throws

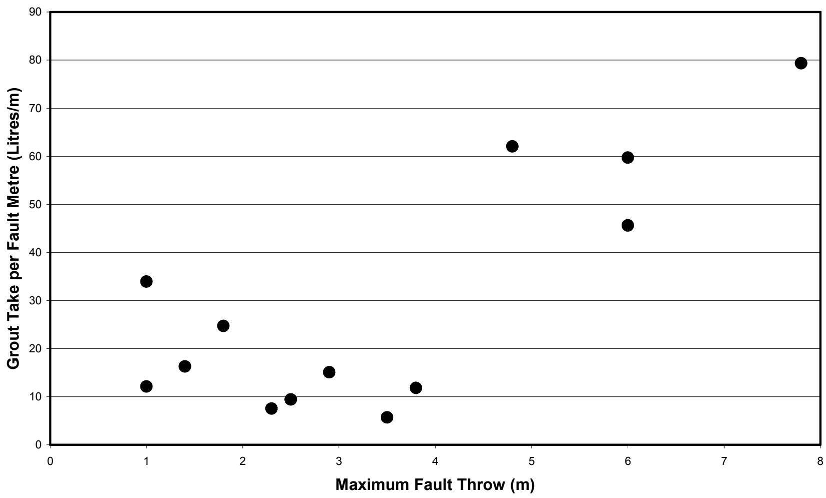

Figure 8 - Comparison between fault throw and grout take in fault

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.

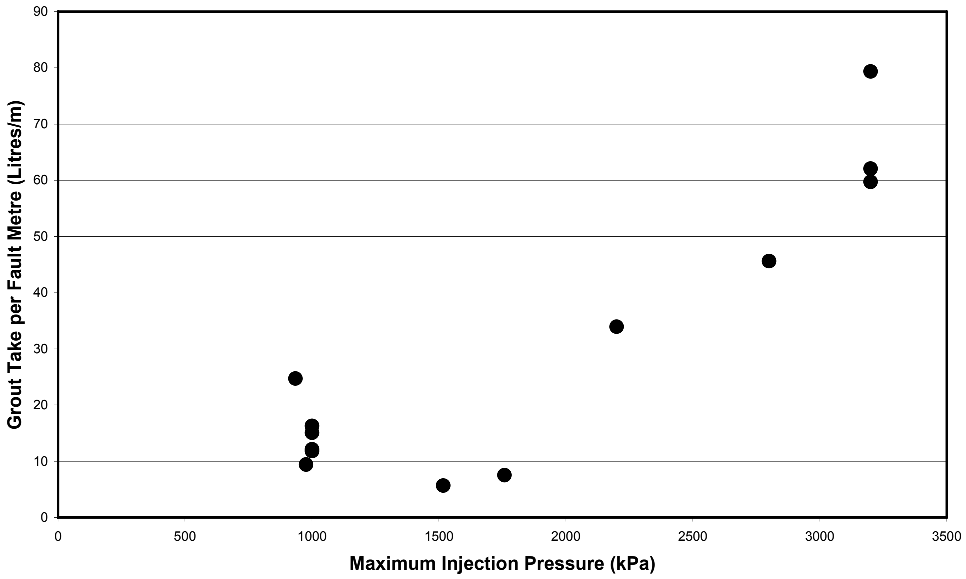

Figure 9 - Relationship between injection pressure and grout take in fault

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.

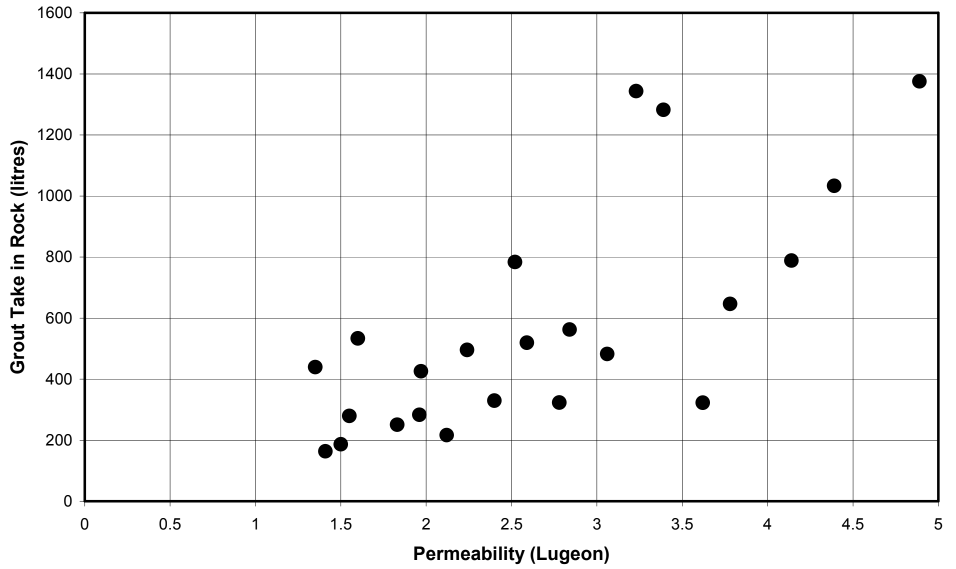

Figure 10 - Typical relationship between permeability and grout take

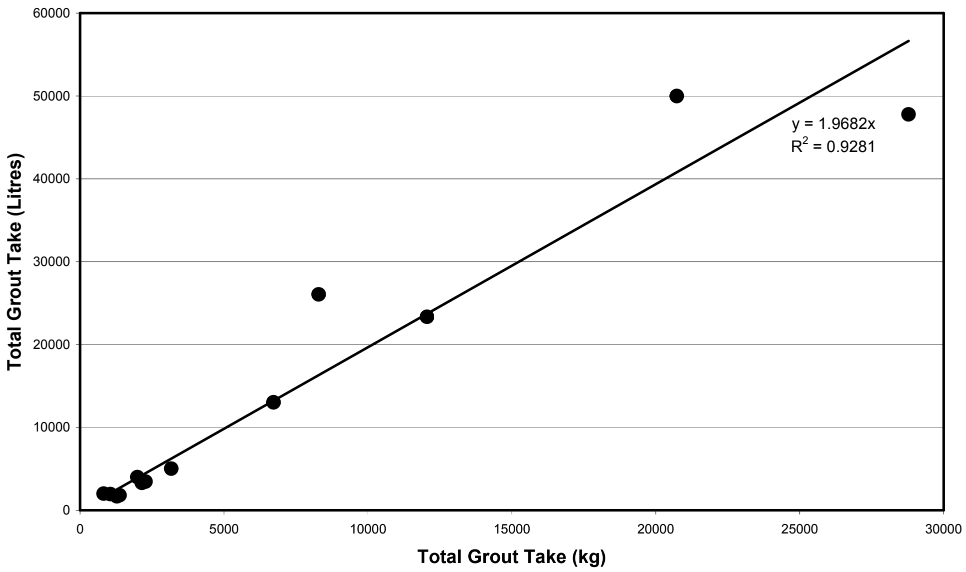

Figure 11 - Comparison between grout volume and cement used

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

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

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[1] and Brett Poulsen[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 asses 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.

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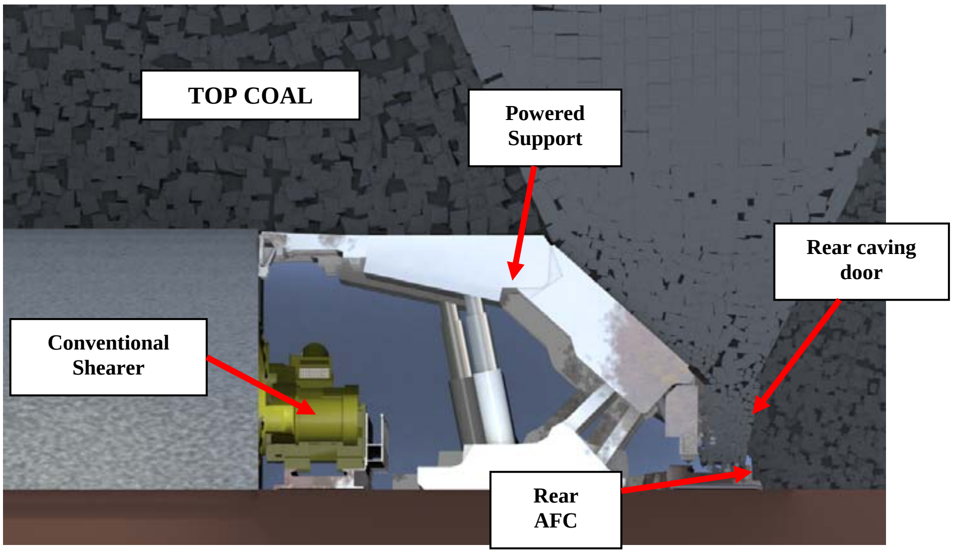

Figure 1 - General LTCC face arrangement

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 conveyer 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 conveyer 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.

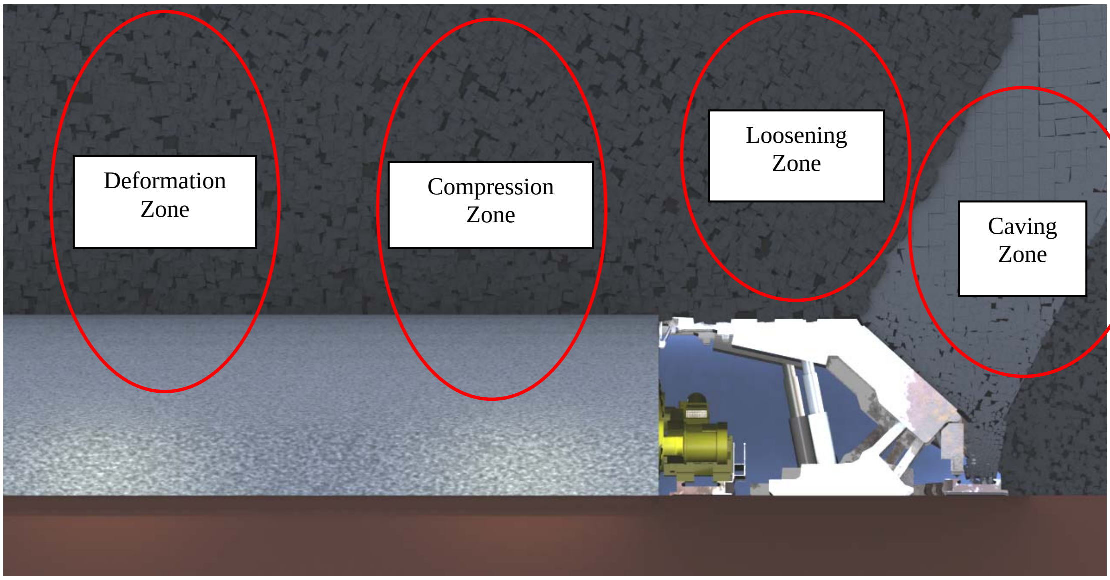

Figure 2 - Top coal fracturing zones

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

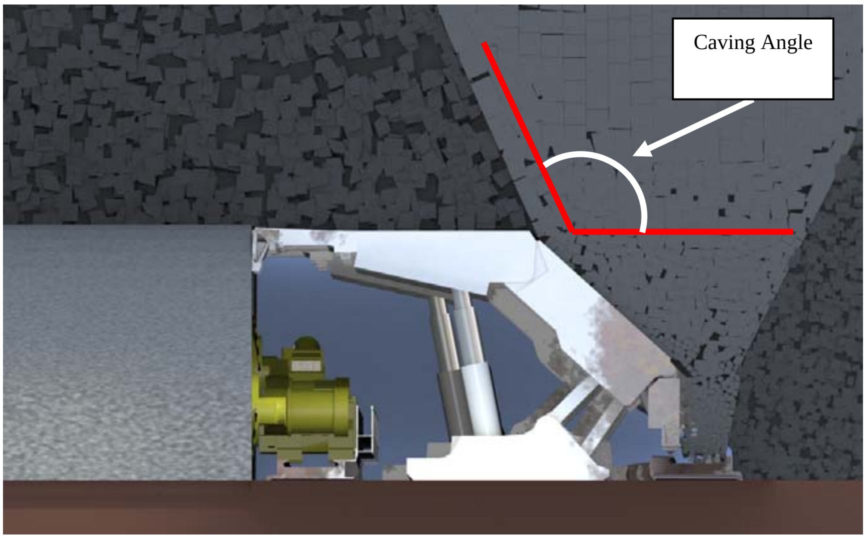

Figure 3 - LTCC caving angle

Current understanding of the interaction between insitu 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:

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

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

LTCC Classification	**1**	**2**	**3**	**4**	**5**
Mining Conditions	Very good	Good	Medium	Bad	Very bad
Caving Index (y)	> 0.9	0.8 – 0.9	0.7 – 0.8	0.6 – 0.7	< 0.6
Top coal Recovery (%)	> 80	65 – 80	50 – 65	30 – 50	< 30

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 insitu 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 insitu 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.

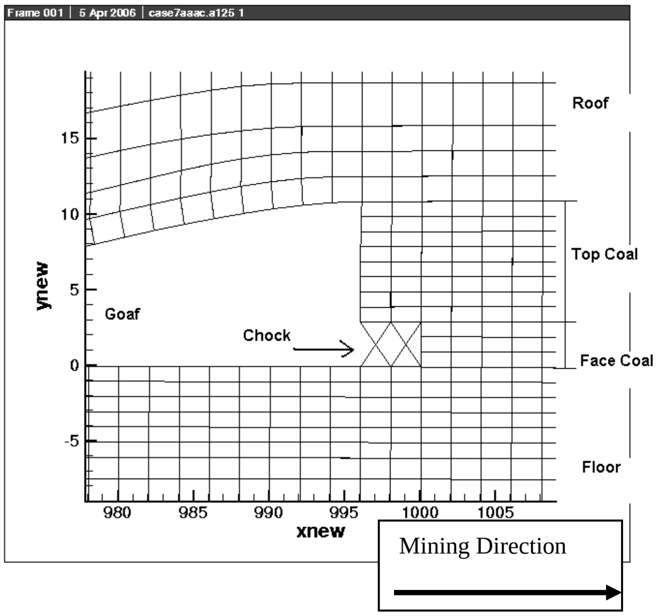

Figure 4 - COSFLOW model for LTCC showing face support (chock) and elements of the numerical mesh. Mining progresses from left to right with face coal removed, chock advanced and top coal removed forming goaf

	Thickness (m)	Strength (MPa)	Notes
final cover	50.0	40.0	elastic only
main roof2	variable	40.0	
main roof 1	20.0	30.0-100.0	
immediate roof	5.0	5.0-30.0	
top coal	3.0-9.0	2.0-10.0	LTCC Mining Horizon
coal	3.0	2.0-10.0	LTCC Mining Horizon
floor	5.0	30.0	
base	200.0	40.0	

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.

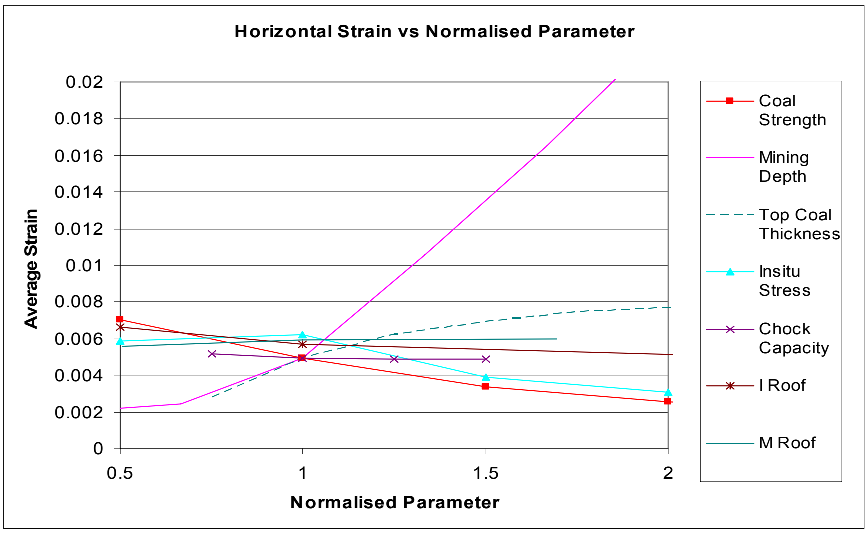

Figure 6 - Results on a common figure by normalising the parameters. Normalising the parameter is done by dividing the variation of the parameter by the base case value. Hence for mining depth where the base case is 300 m, 150 m is normalised to 0.5 and 600 m to 2.0.

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:

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

Where CI is here after 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:

H = 3.08
CS = 0.54
TC = 0.47
CC = 0.22
IR = 0.15
MR = 0.008

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 multi variate regression was undertaken to develop a new simplified equation.
Figure **7** shows a plot of the three parameters.

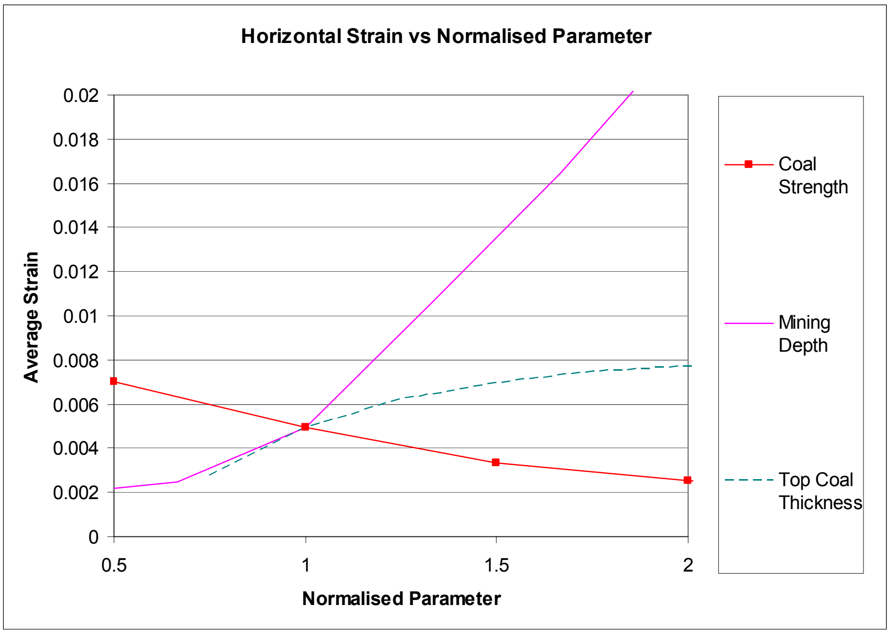

Figure 7 - Three most important parameters as determined from the standard regression coefficients

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.

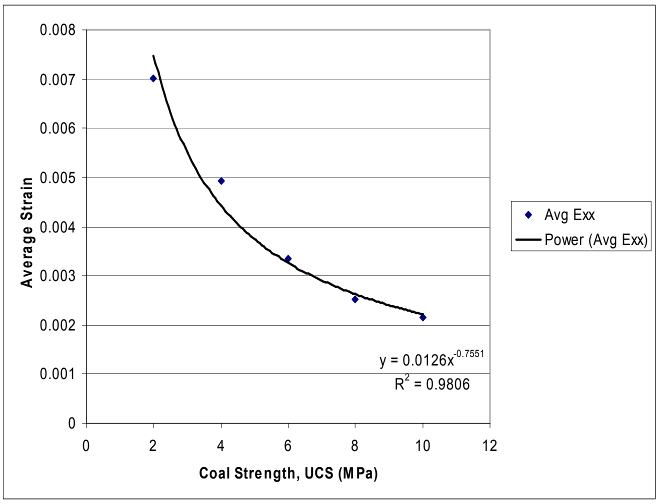

Figure 8 - Plastic strain verse coal strength

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.

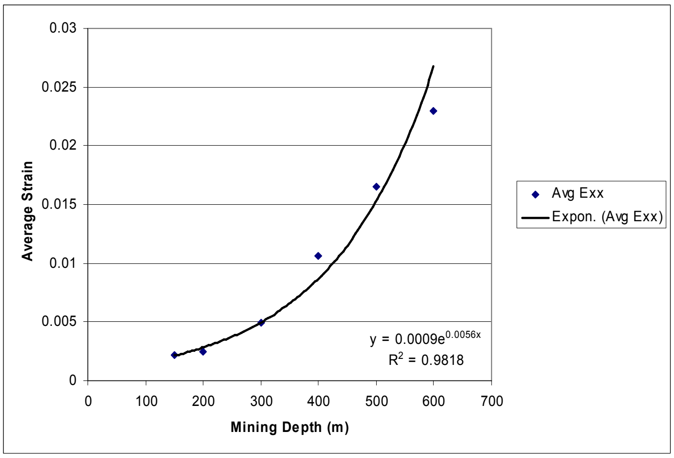

Figure 9 - Plastic strain verse mining depth

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.

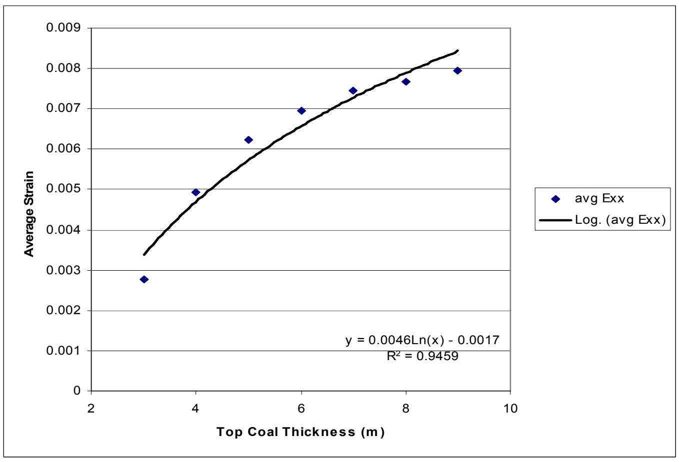

Figure 10 - Plastic strain verse top coal thickness

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:

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

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**.

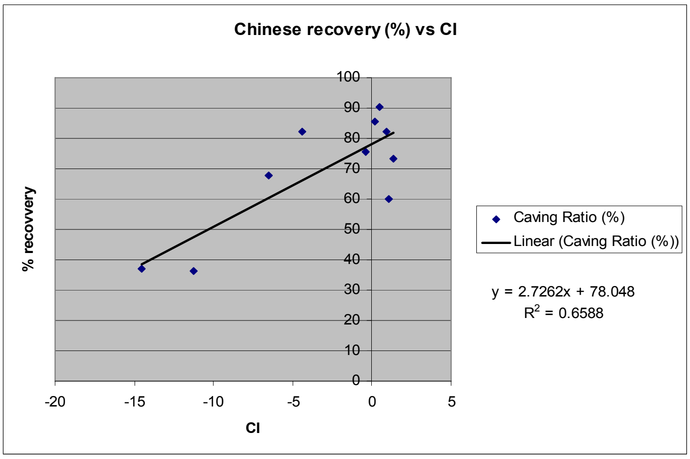

Figure 11 - Chinese mine recoveries plotted against COSFLOW CI equation

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:

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

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

$$\mathbf{CI = -2.64 + 0.0395 H - 0.72 CS + 0.191 TC} \qquad \mathbf{[5]}$$

where;

CI is the Caving Index

H= mining depth (m)
CS= uniaxial coal strength (MPa)
TC= top coal thickness (m)

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:

$$\text{Percentage top coal recovery} = 2.72\ \text{CI} + 78.0$$

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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THE LIMITATIONS OF THE OBSERVATIONAL METHOD AND MONITORING PROGRAMS FOR HIGH PRODUCTION LONGWALLS AND AN ALTERNATIVE FRAMEWORK

Ross Seedsman[1]

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.

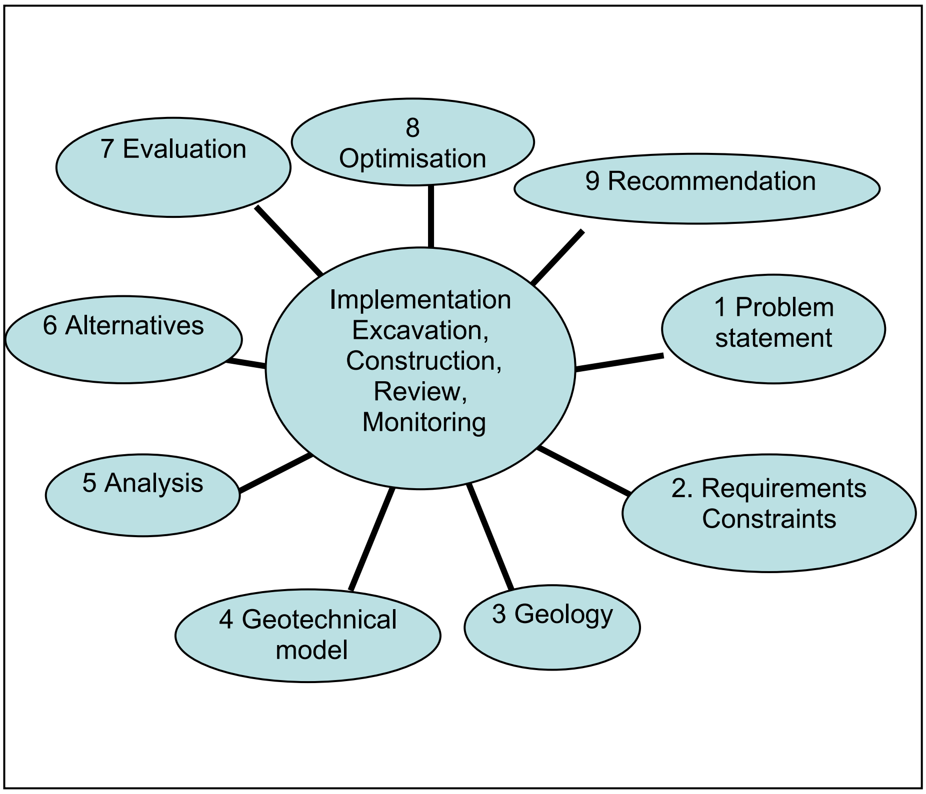


Figure 12 - A rock mechanics design methodology

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.

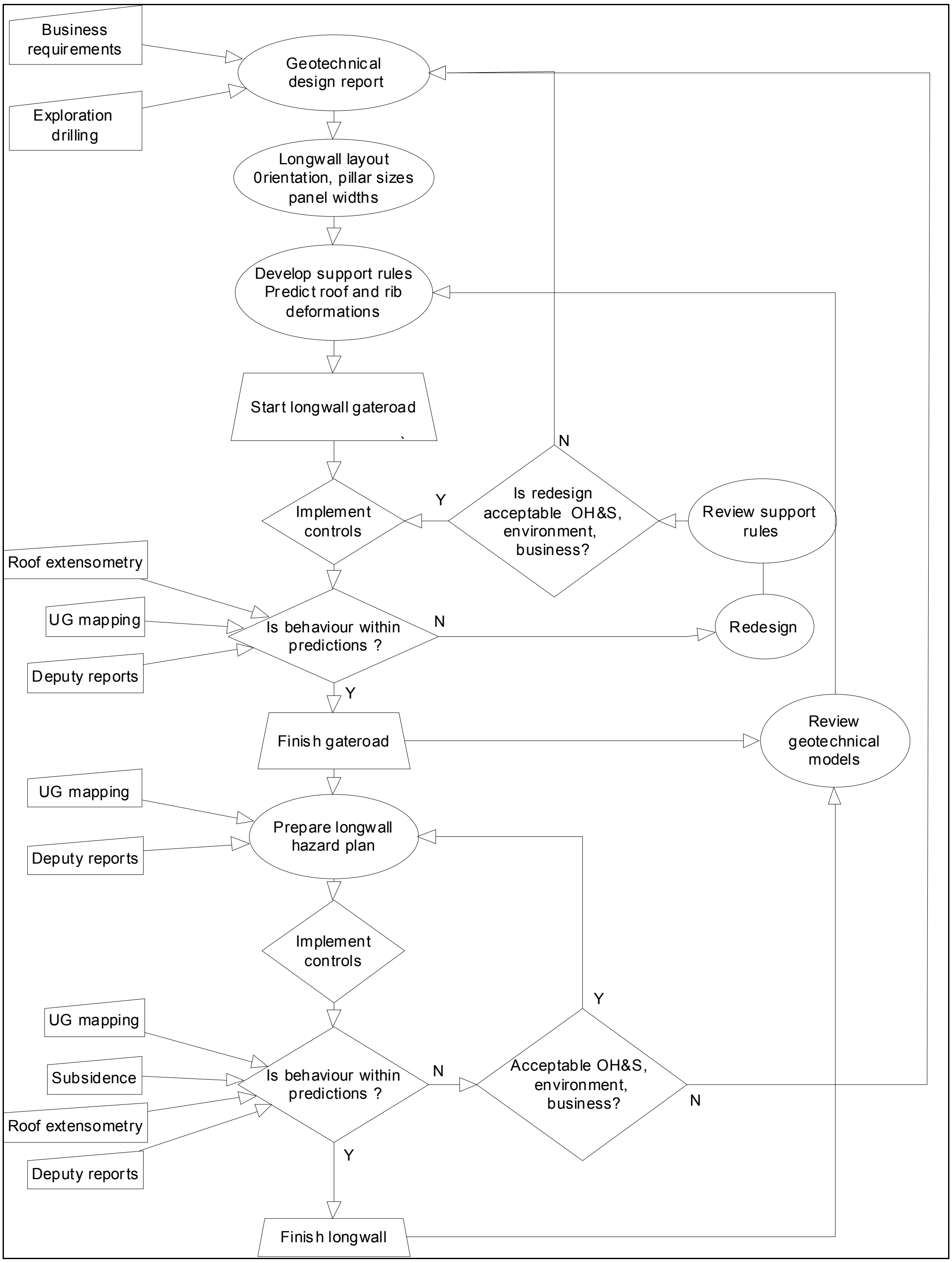


Figure 2 - A model for the integration of geological and geotechnical programs into longwall design, planning, and operations (diamonds – SMT, ovals – geotechnical designer)

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.

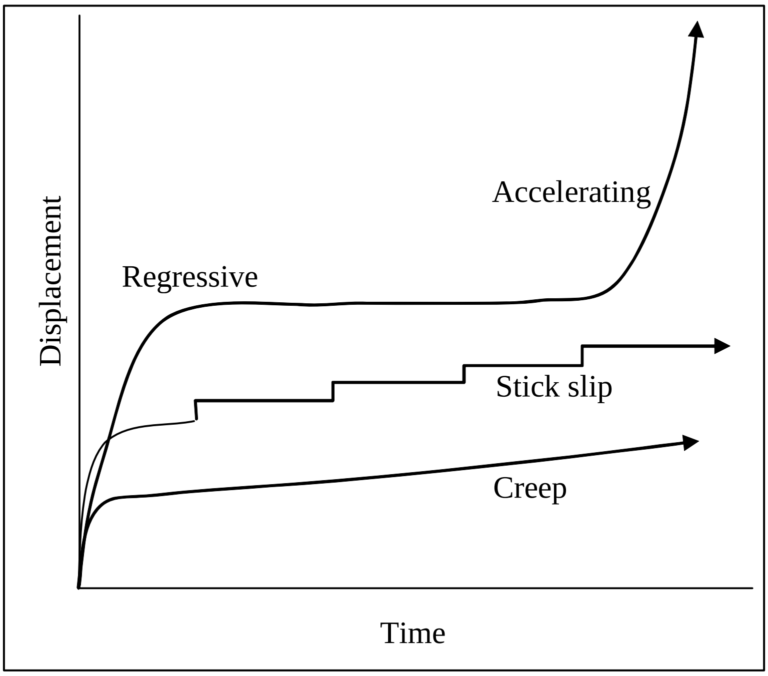


Figure 3 - Different patterns of roof movement

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 predictins 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."

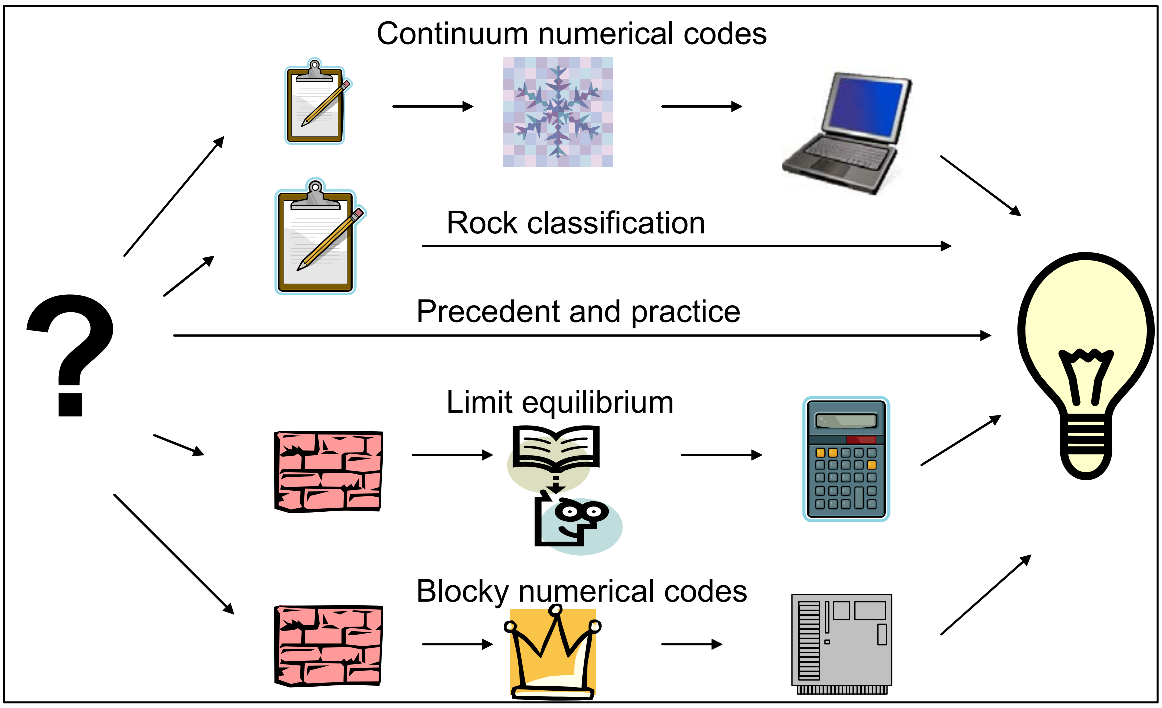


Figure 4 - Cartoon showing 5 different approaches to design in rock mechanics

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.

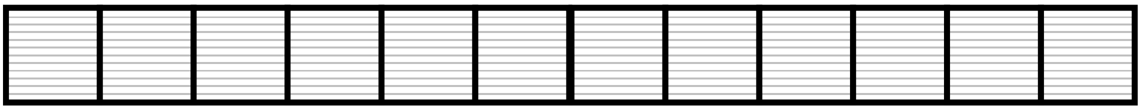
The limit equilibrium method is not well developed. In recent years it has often been considered as unnecessary in the coal sector in the face of sophisticated numerical codes. The author is currently developing an approach for underground coal. Coal mine geometry is much simpler than the typical metal 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 a assemblage of rectangular prisms (Figure 5) include:

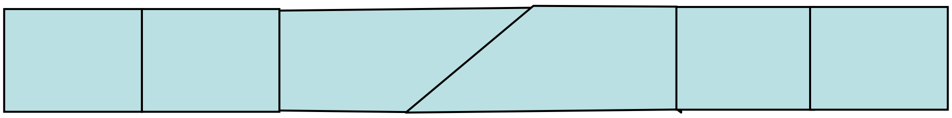
- Compressive failure of the rock substance if the lateral stress is compressive and the deviatoric stresses in the roof exceeds the compressive strength
- Gravity fall of joint blocks if the lateral stress is tensile
- Delamination/buckling along bedding partings under self weight and imposed compressive 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.



(a) Bedded jointed rock with finite compressive strength



(b) Low angle surface means roof collapses in all stress conditions

Figure 5 - Collapse modes for coal mine roof

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.

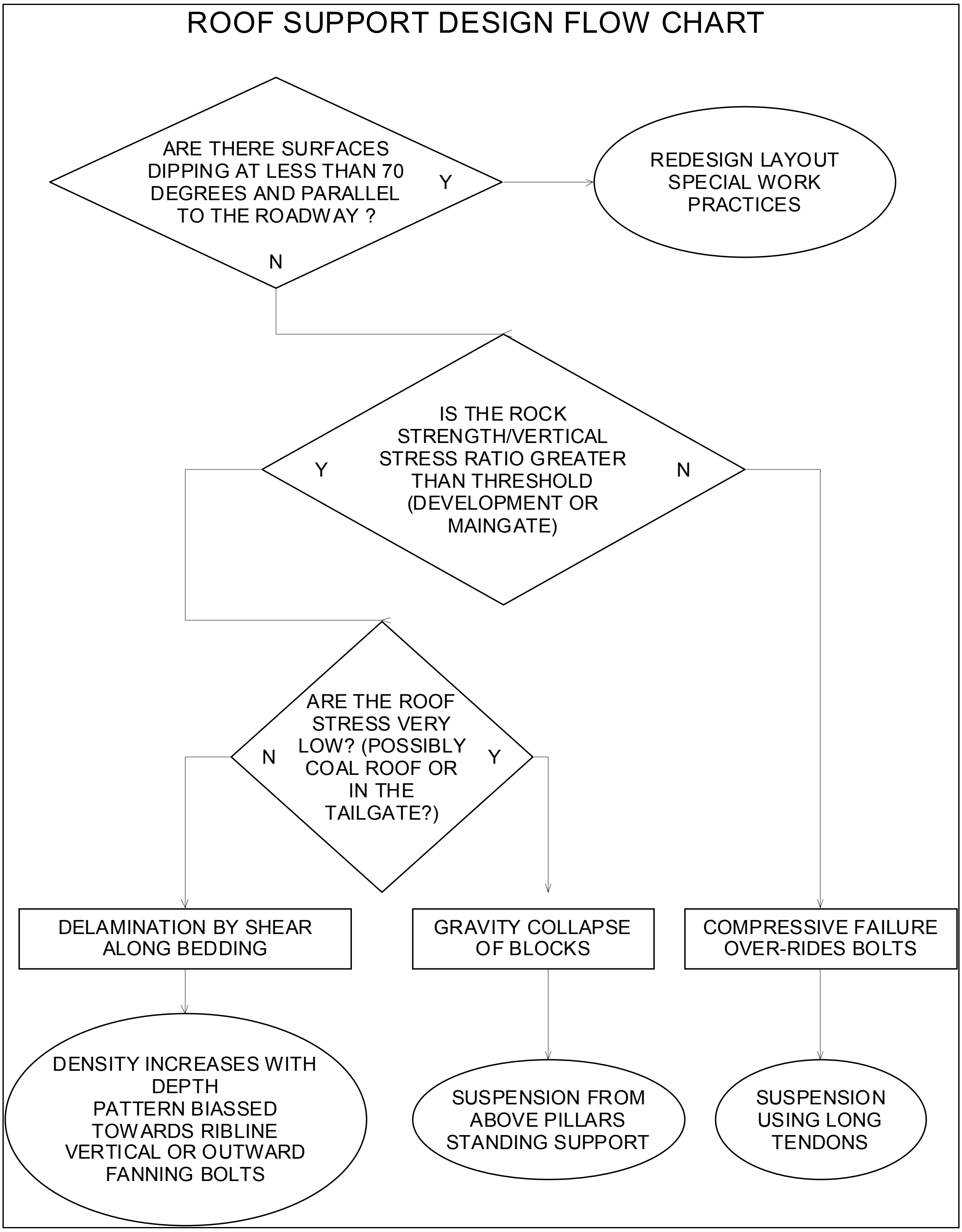


Figure 6 - Logical framework for coal mine excavation design

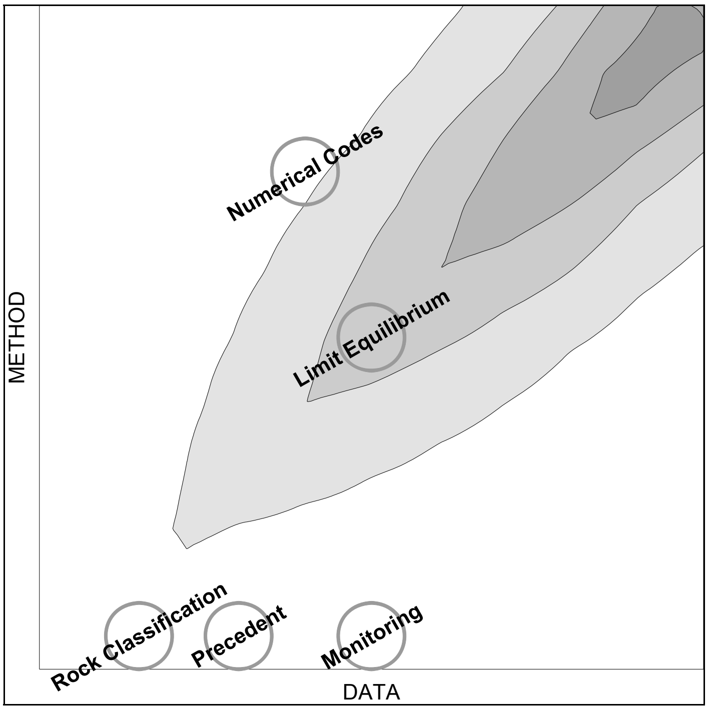


Figure 7 - Design can be limited by the method or the data

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TAILGATE 802 - GRASSTREE MINE: A CASE STUDY IN PRAGMATIC ROADWAY ROOF SUPPORT DESIGN

Mark Colwell[1], Russell Frith[2], Guy Reed[3]

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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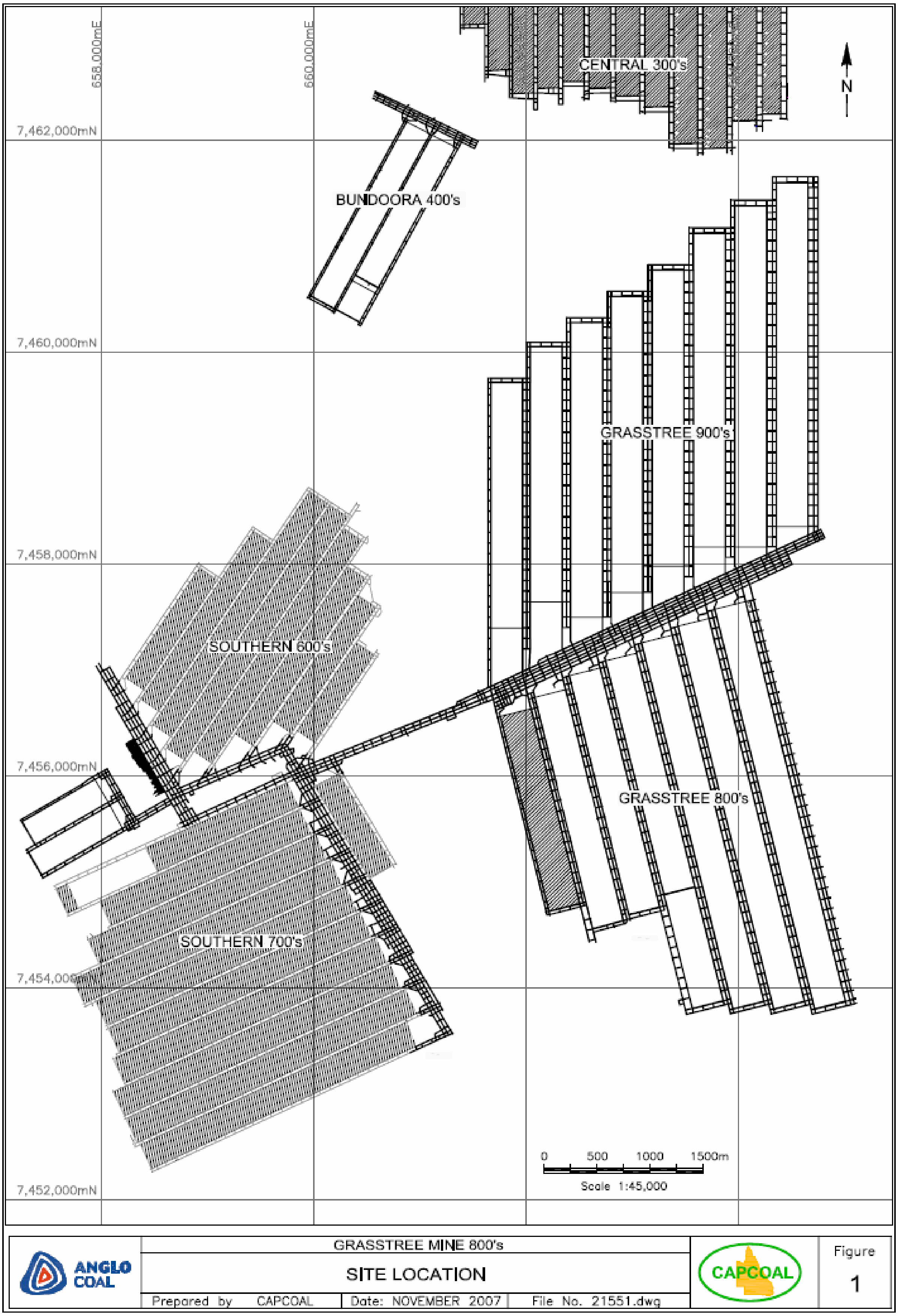

Figure 1 - Site location

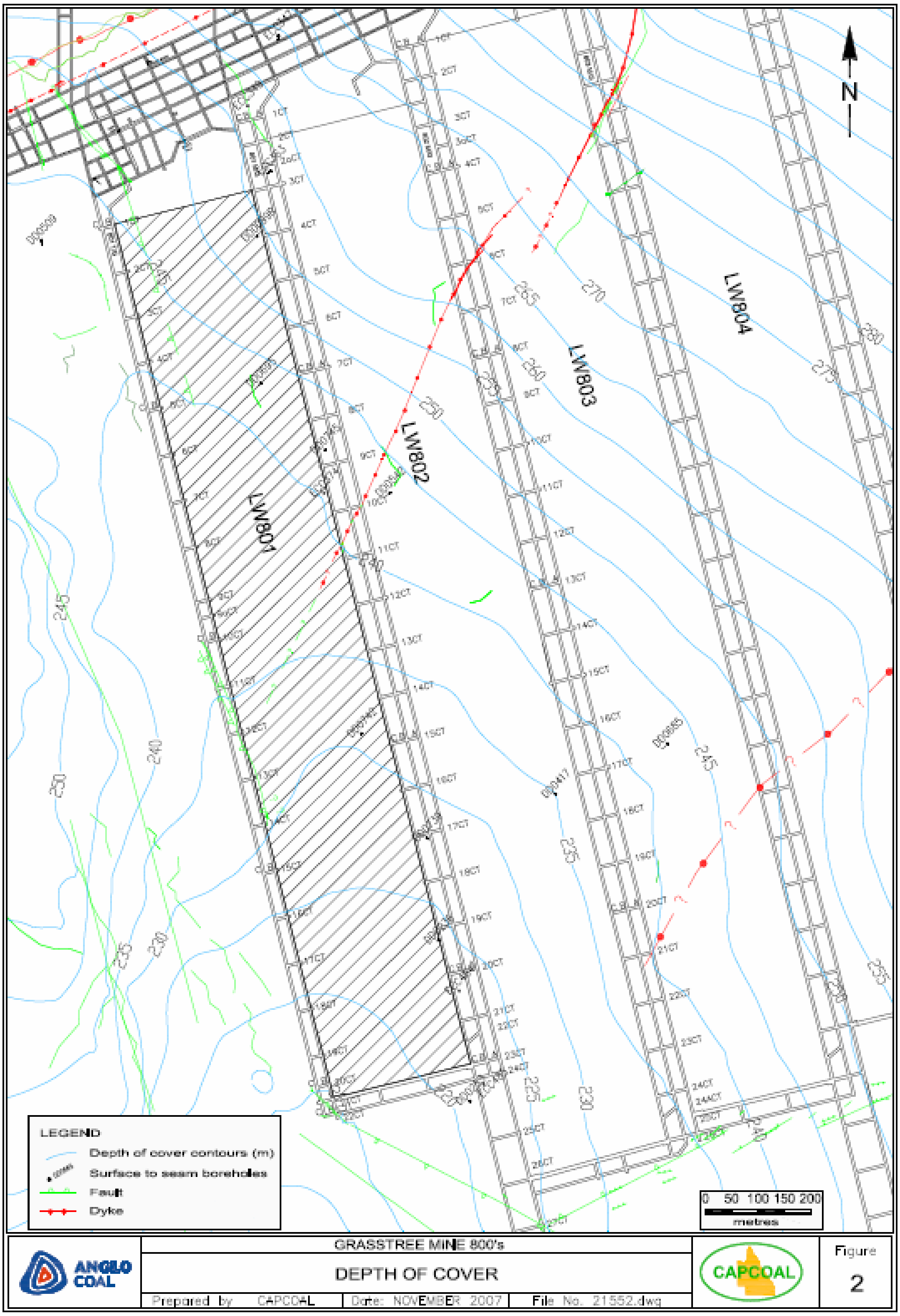

Figure 2 – Depth of cover

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:

$$\text{ALPS } SF_R = 1.76 - 0.014 \text{ CMRR} \qquad (1)$$

$$\text{TG } SF_R = 2.881 - 0.0343 \text{ CMRR} \qquad (2)$$

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:

$$\text{Equivalent TG SF} = \text{Calculated ALPS SF} \times (\text{TG } SF_R/\text{ALPS } SF_R) \qquad (3)$$

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.

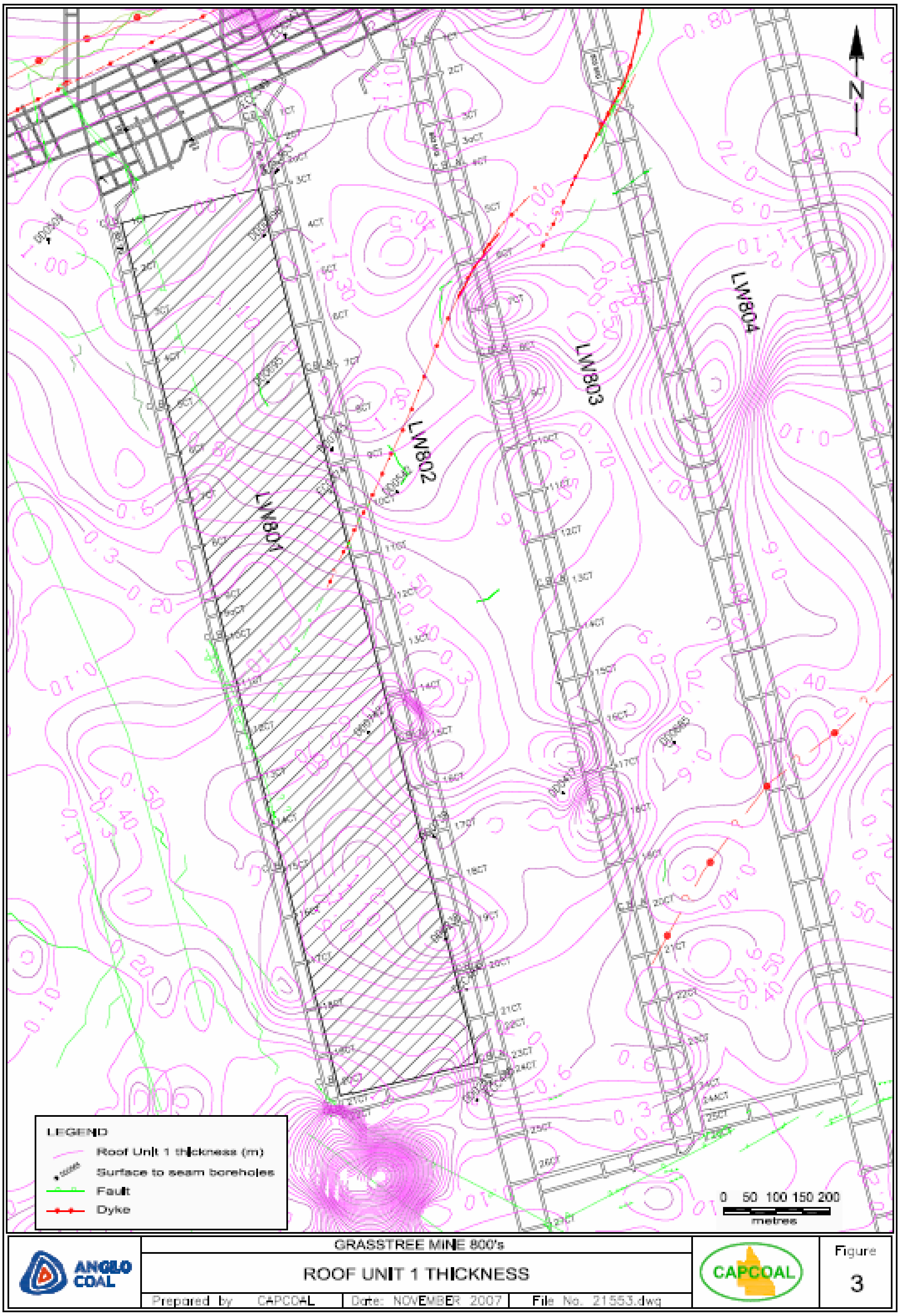

Figure 3 – Roof unit 1 thickness

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 ($I_{s(50)}$) 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 $I_{s(50)}$, 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 $I_{s(50)}$ 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 laminated ROF1 for those boreholes where Diametral $I_{s(50)}$ 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 $I_{s(50)}$ 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 $I_{s(50)}$ and ROF2's Fracture Spacing.

Table 1 - Summary of CMRR calculations for 1.8m bolt length

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

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 $I_{s(50)}$ 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 $I_{s(50)}$ 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 $I_{s(50)}$ 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 ≤ 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 ≥ 1m; 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 ≤ 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 ≤ 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:

$$\text{FOS} = \text{load bearing ability/applied load} \qquad (4)$$

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.

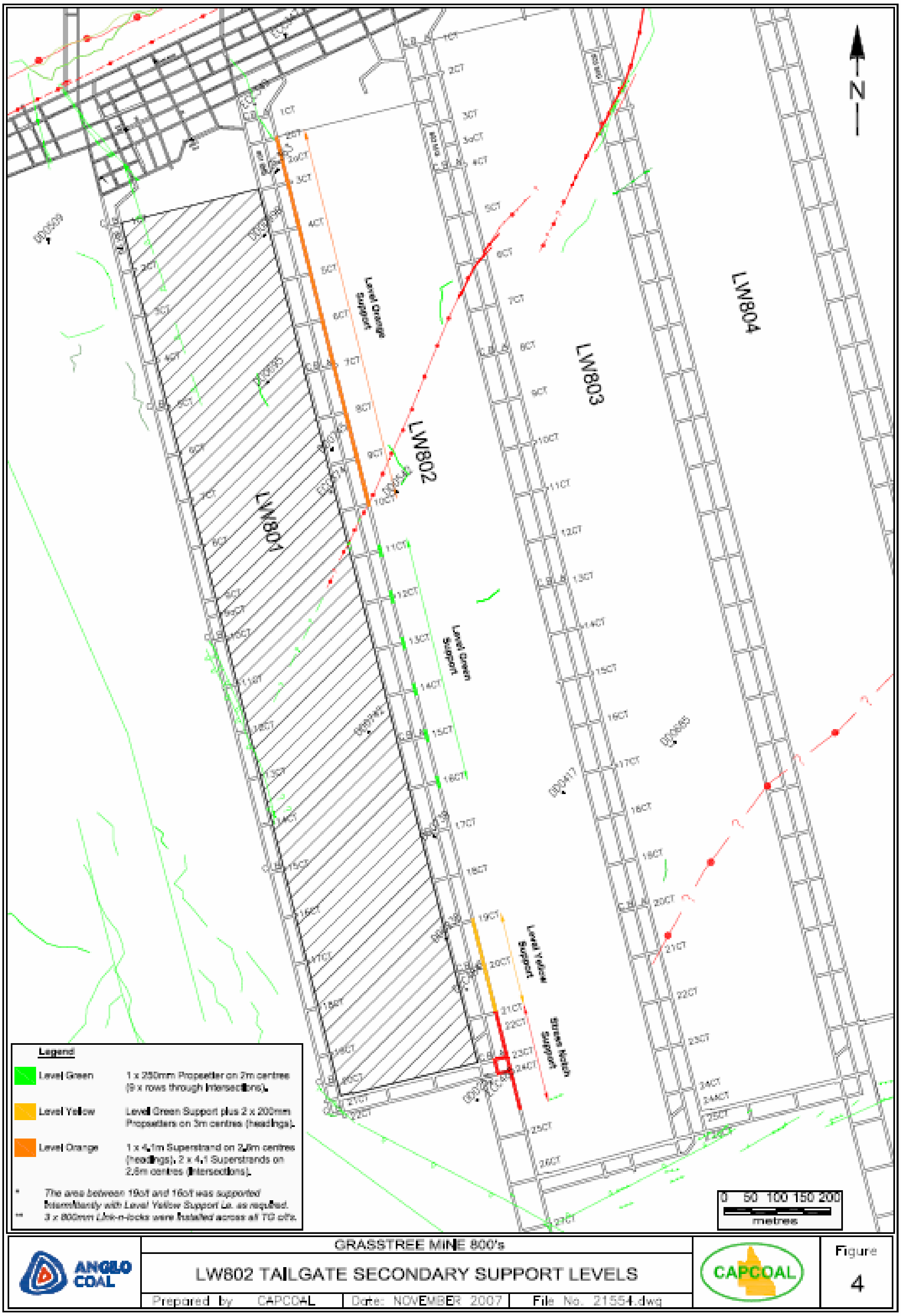

Figure 4 – LW 802 tailgate secondary support levels

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 site 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 re-written as:

FOS = load bearing ability (roof strata + roof support)/applied load (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_{roof}) and subsequently determining the lateral resistance offered by the roof support ($P_{support}$). 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:

$$\sigma_H = \nu/(1-\nu).\ \sigma_V + TSF \times E \qquad (6)$$

where:

ν = Poisson's Ratio
$\nu/(1-\nu) = K_o$
E = Young's Modulus (in GPa)
σ_V = vertical stress acting (where σ_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:

$$\sigma_H = 5.75 \times 10^{-3}\ H + 11.04\ (MPa) \qquad (7)$$

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 *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 *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 *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 *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_{roof})

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_{roof}) can be deduced.

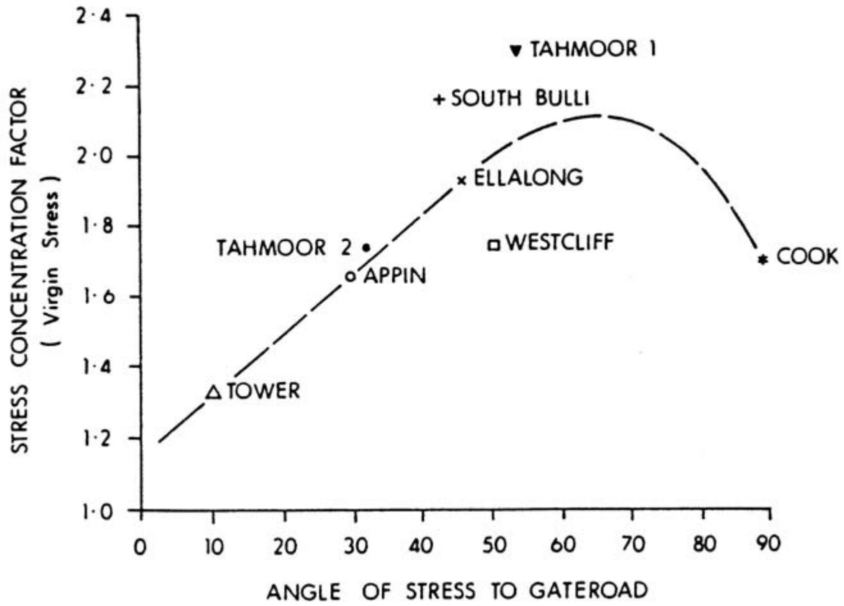

Figure 5 - Relationship between horizontal SCF and angle of gateroad to stress direction (after Gale and Matthews 1992)

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).

$$\sigma_{crit} = \pi^2 E /[12(L_{eff} /d)^2] \qquad (8)$$

Where E is Young's Modulus, L_{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 (σ_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/statics/Columns/cols62.htm.

The Johnson equation for the allowable stress is as follows:

$$\sigma_{crit} = [\, 1 - (L_{eff} / ŕ)^2/(2\, C^2)]\, \sigma_y \qquad (9)$$

Where ŕ is beam's Radius of Gyration and C is the beam's Critical Slenderness Ratio

$$ŕ = I/A \text{ and } C = (2\, \pi^2\, E/\sigma_y)^{0.5}$$

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.*

Essentially when the beam's Slenderness Ratio (L_{eff} /t) 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 σ_y (where σ_y = 0.7 x UCS) of the rock unit and the beam's effective length (L_{eff}) 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 (σ_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_{eff} 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 x 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 1m^2 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.

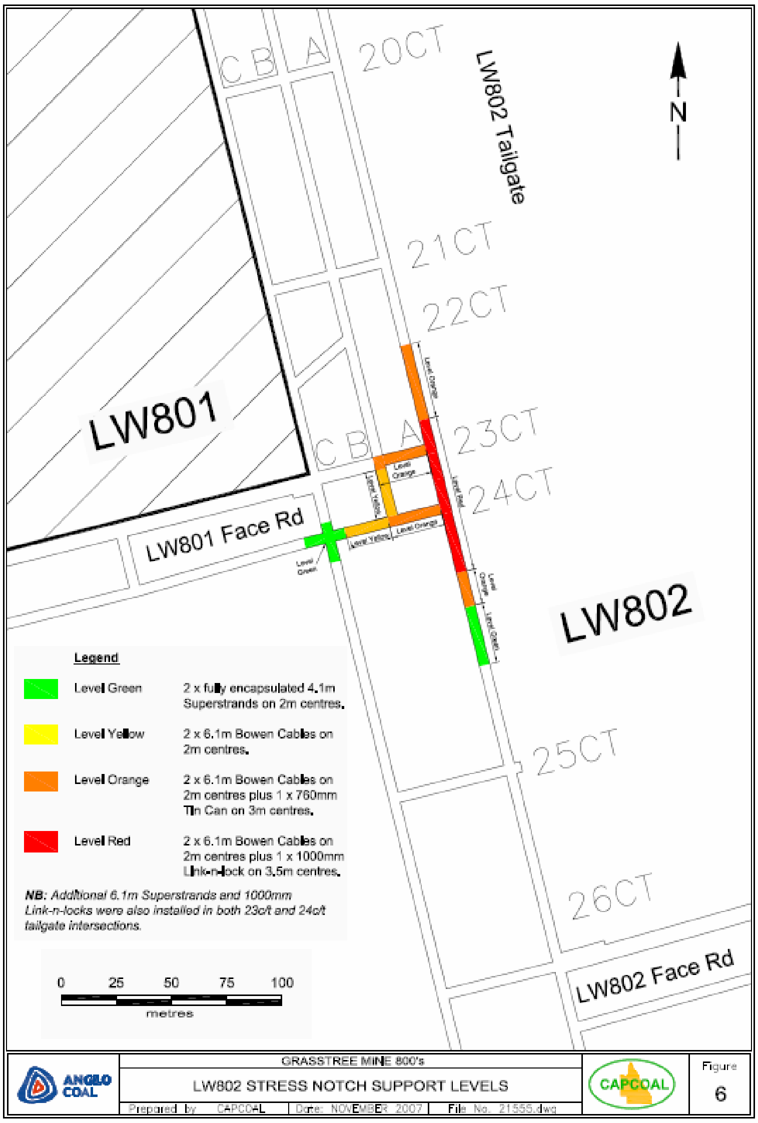

Figure 6 –LW802 stress notch support levels

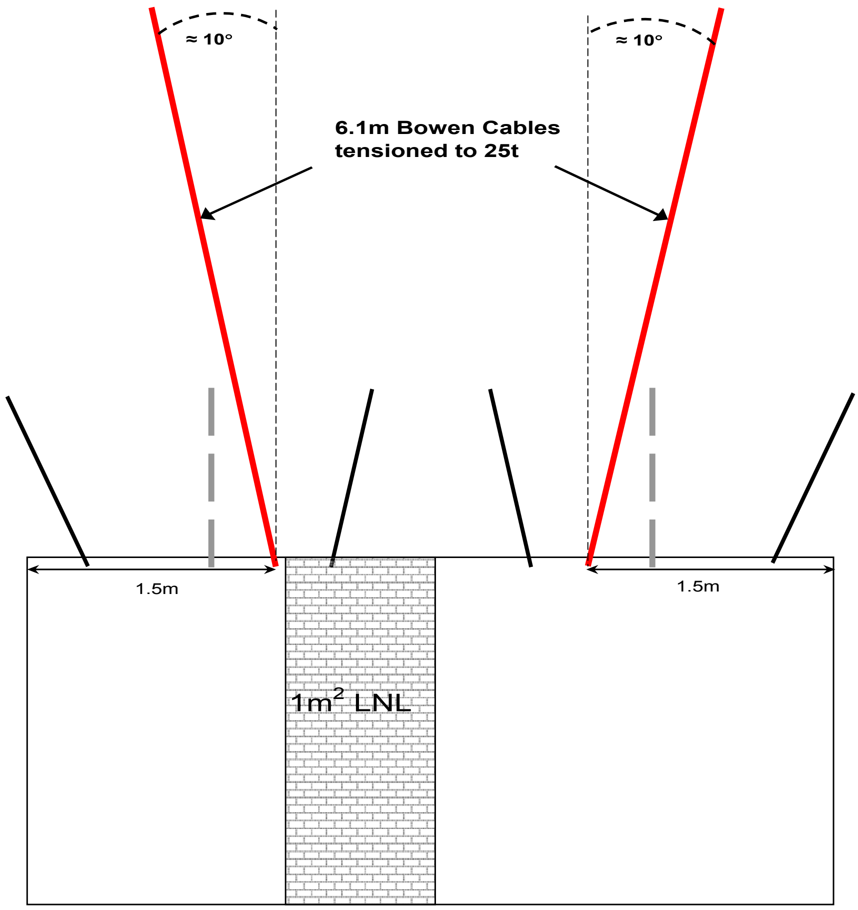

Figure 7 - Roof support installed within the super stress notch zone

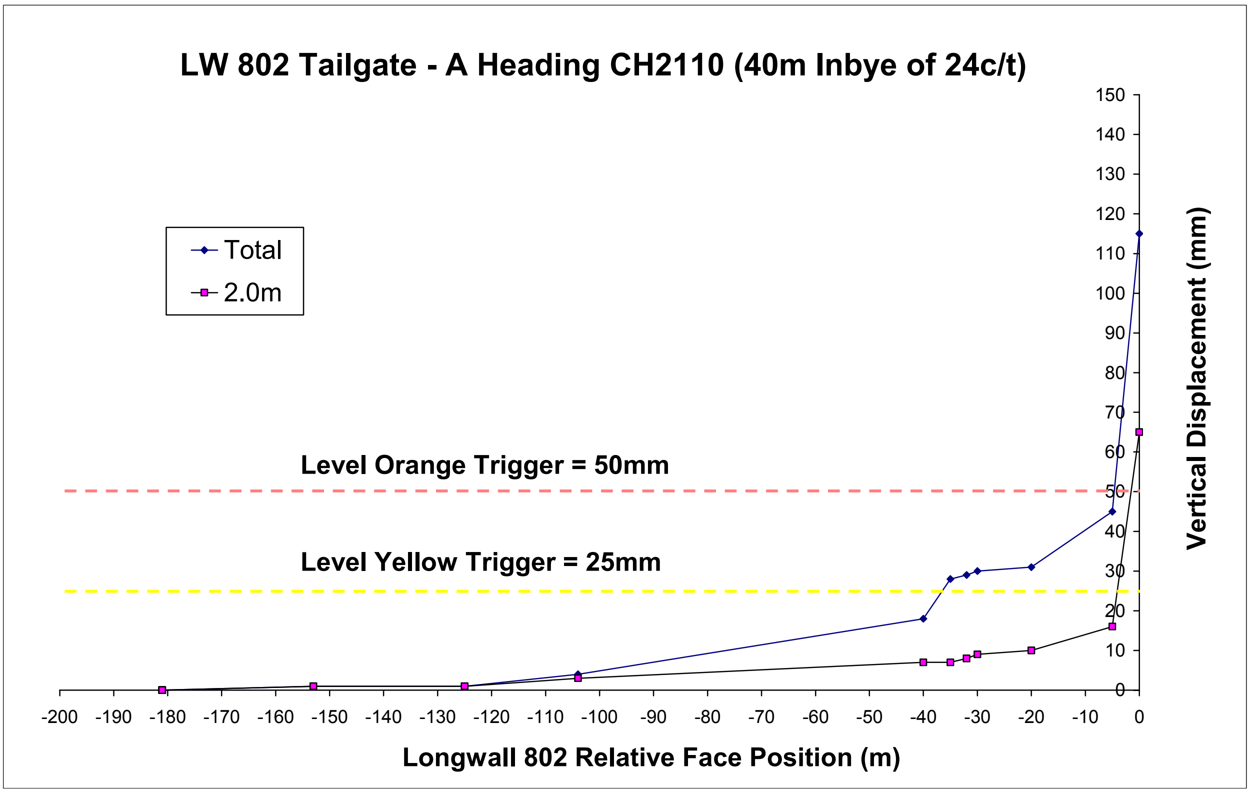

Figure 8 - Tell tale data associated with the super stress notch zone

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 minesite Strata Control/Geotechnical Engineers. This is a primary goal of a current industry sponsored project.

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CMRR – PRACTICAL LIMITATIONS AND SOLUTIONS

Justine Calleja[1]

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

Figure 2 - Core photograph

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.

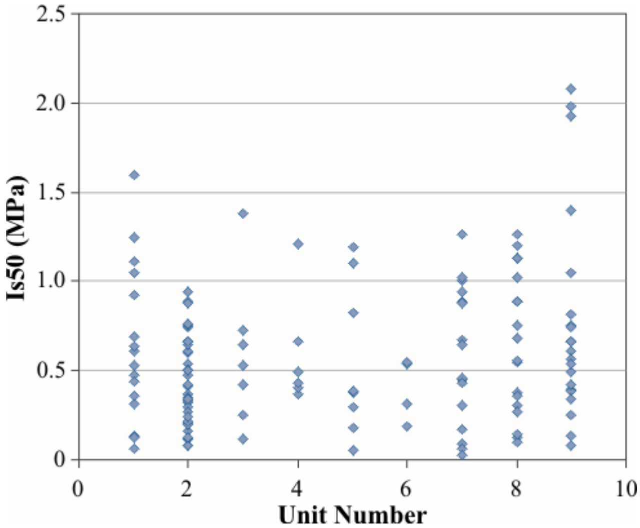

Figure 3 - Diametral pointload test Is50 by unit

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.

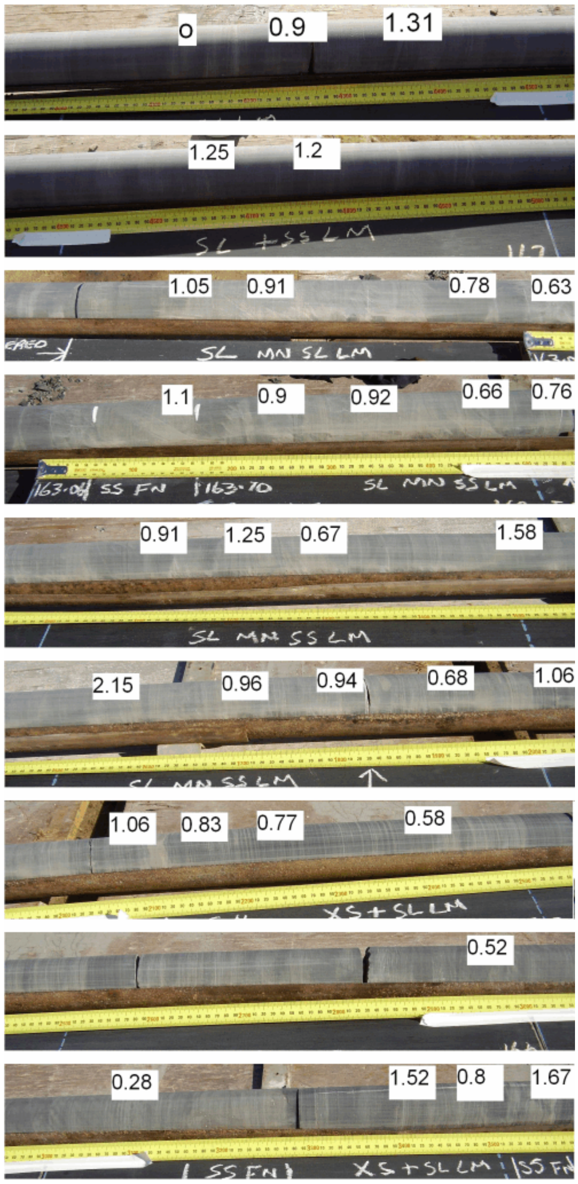

Figure 4 - Core photographs showing diametral PLT Is50 values

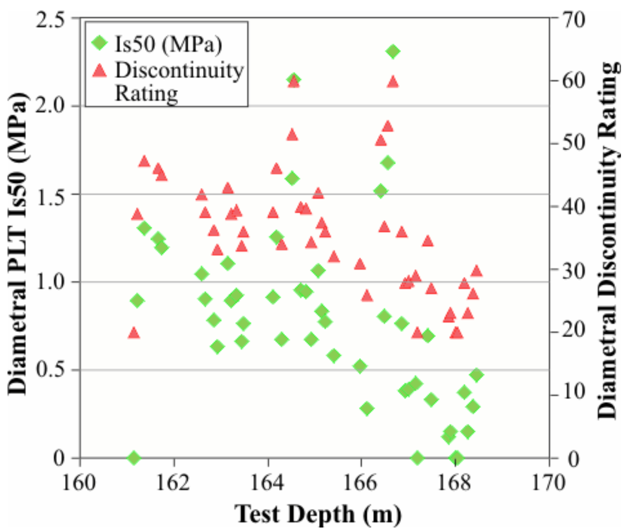

Figure 5 - Diametral PLT values and discontinuity ratings versus depth

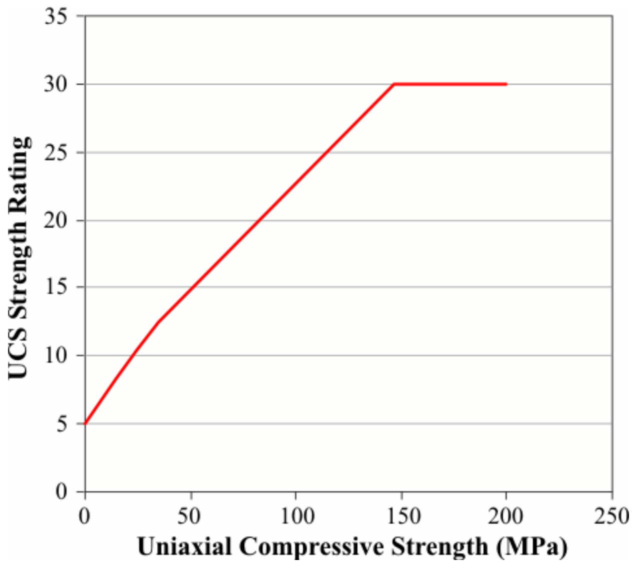

Figure 6 - UCS strength ratings

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).

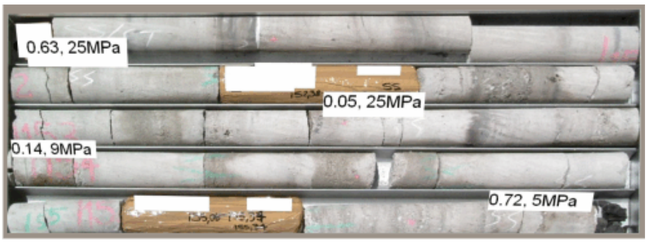

Figure 7 - Core photograph showing Axial PLT, inferred UCS and Diametral PLT Is50 values

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).

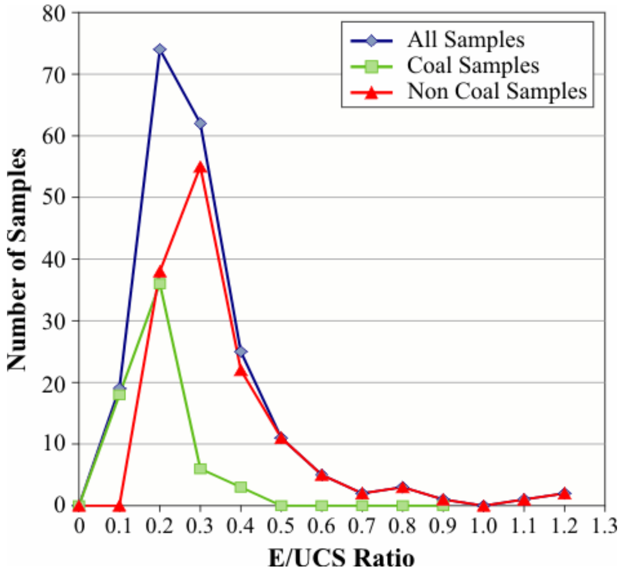

Figure 8 - E/UCS ratio for all samples.

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.

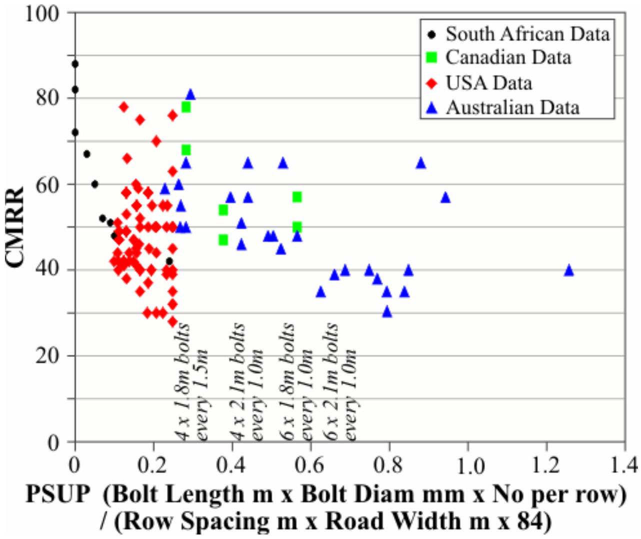

Figure 9 - PSUP (primary support density) versus CMRR.

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 it 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 Frith[1] and Mark Colwell[2]

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.

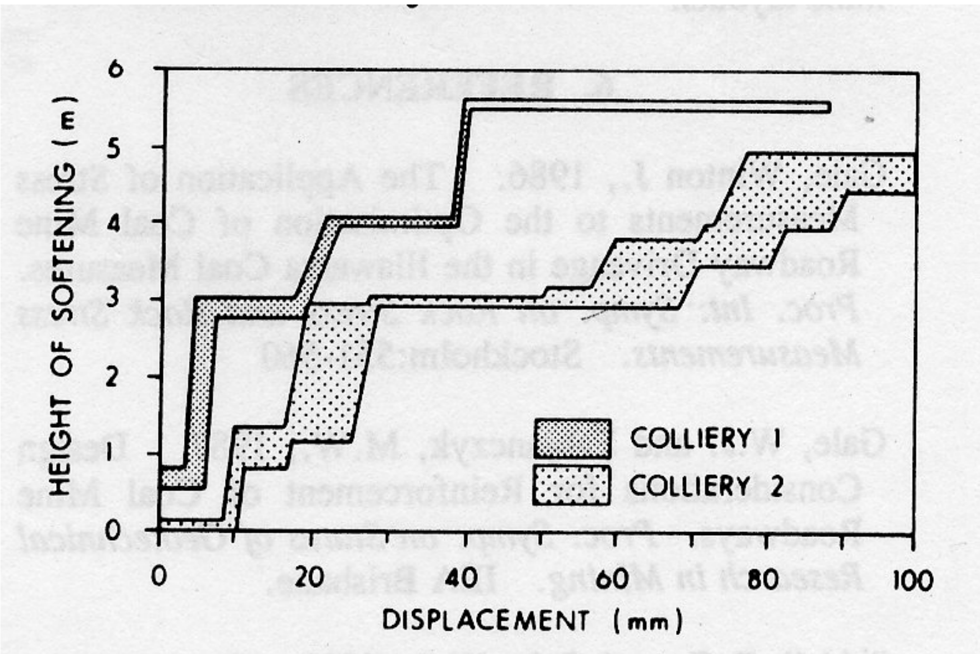

Figure 1 - Roof softening progression with displacement (Gale et al 1992)

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.

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

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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.

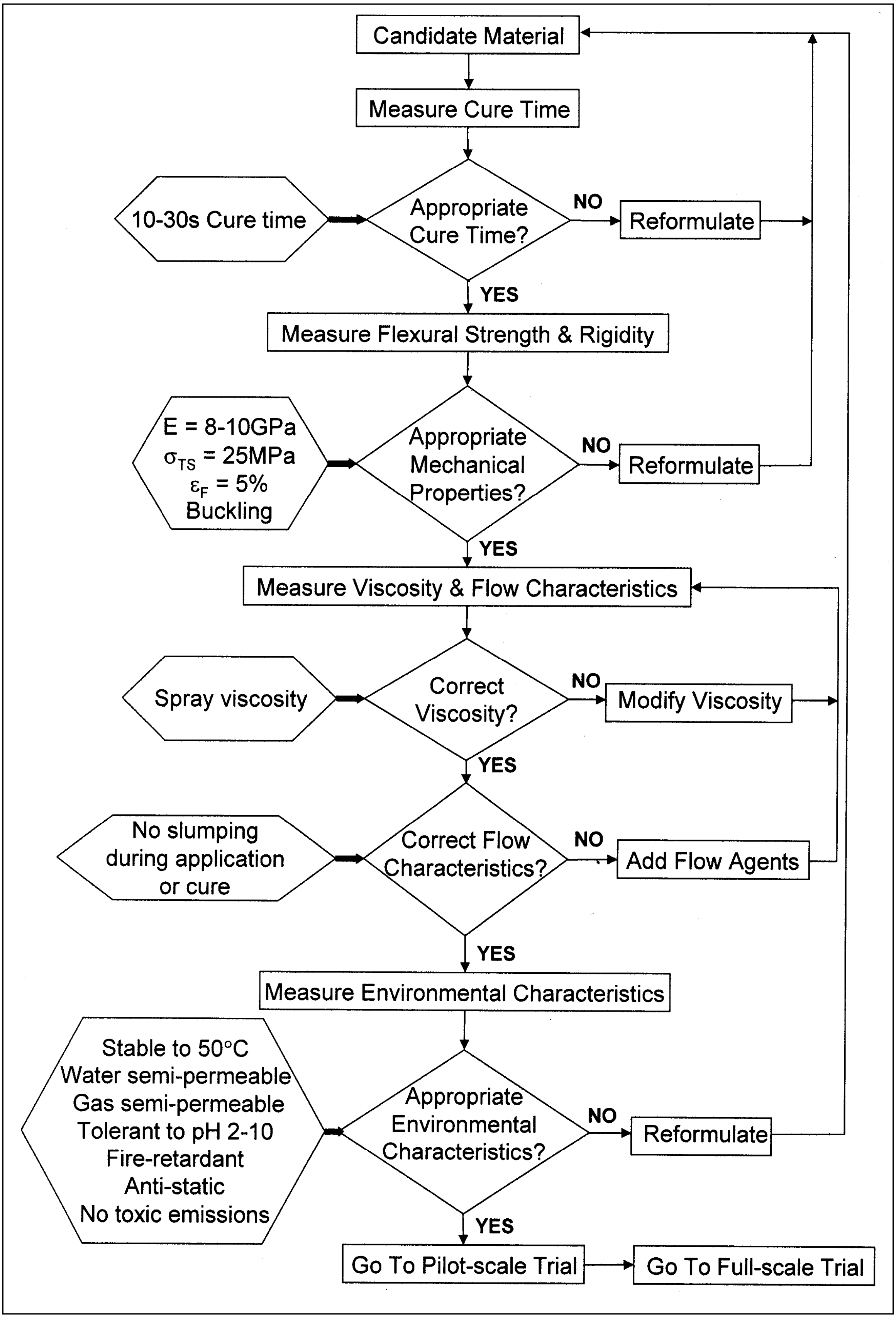

Figure 1 - Flowchart for selection of candidate materials

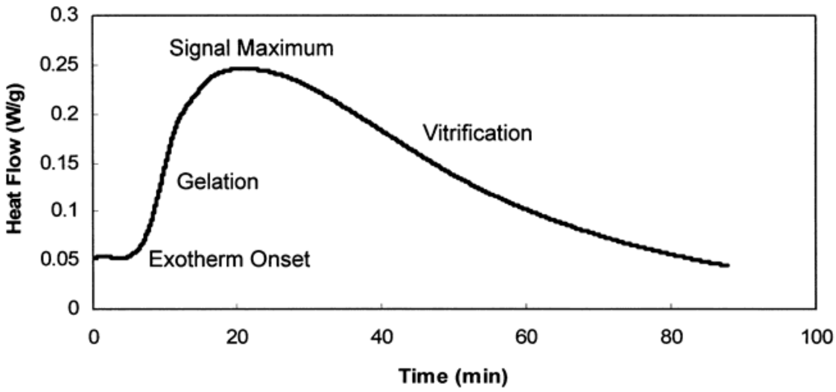

Figure 2 - Typical DSC measurement of crosslinking polymer cure

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 σ_{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 (δ) 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.

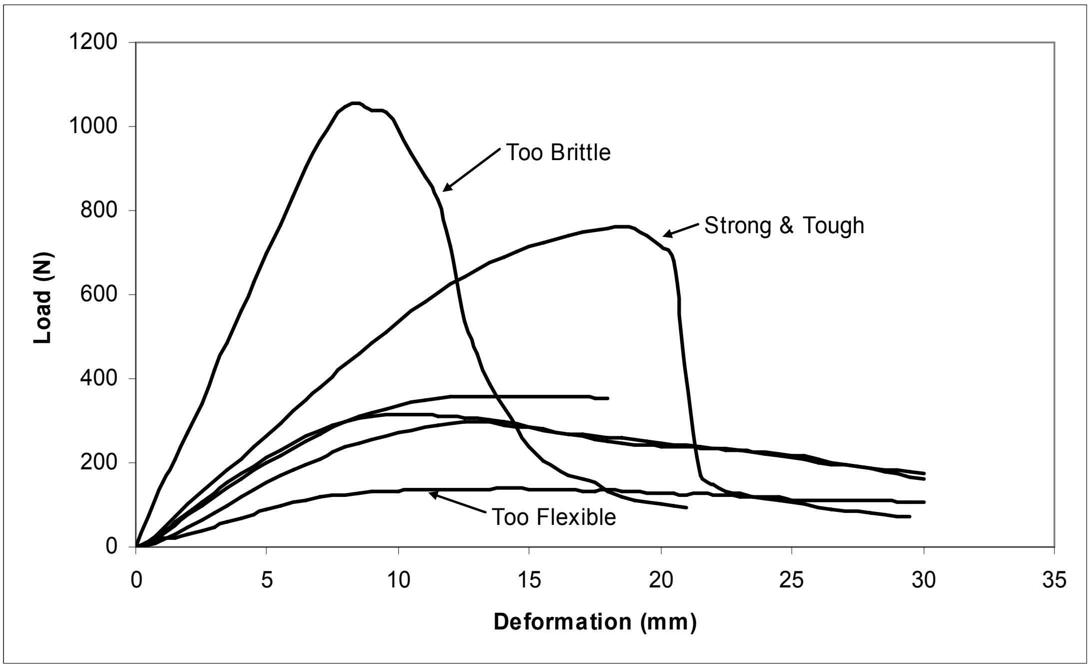

Figure 3 - Flexure Tests on Candidate Reinforced Polymers

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

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

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

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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 affect 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 1 - The number of tests performed in each test category.

Type of bolt	**Type of confining pipe**		
	Steel	Aluminium	PVC
CT bar Φ28 with Epoxy	4	4	4
CT bar Φ 28 without Epoxy	3	3	3
Rebar Φ28 with Epoxy	4	4	4
Rebar Φ28 without Epoxy	3	3	3

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_{rm}) equation (1) can be utilized.

$$k_r = \frac{2E}{(1+\nu)}\left[\frac{d_o^2 - d_i^2}{d_i\left[(1-2\nu)d_i^2 + d_o^2\right]}\right] \qquad (1)$$

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.

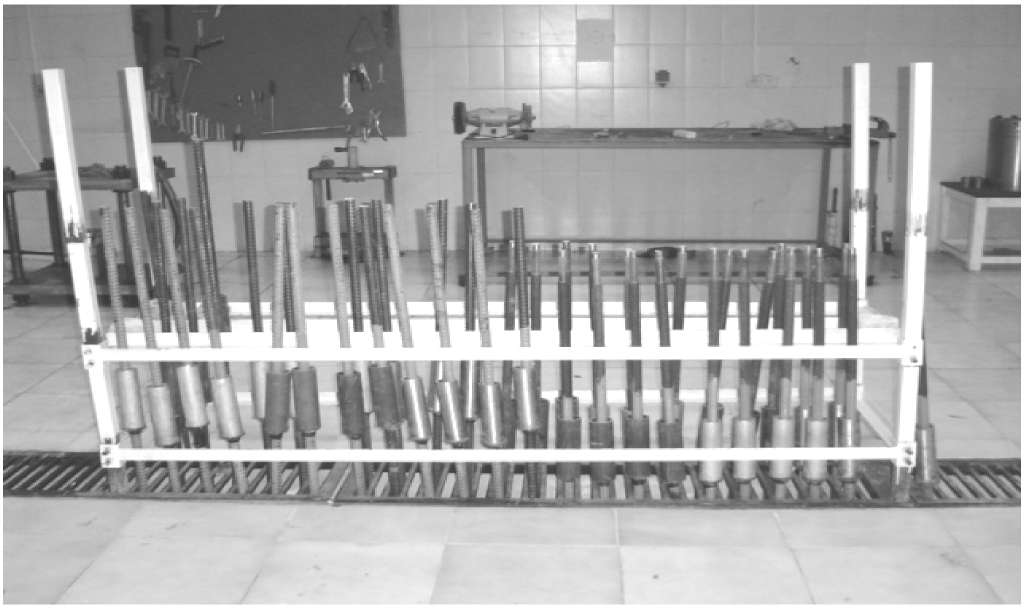

Figure 1 - Sample preparation.

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

	E (GPa)	v	d_o (mm)	d_i (mm)	k_r (MPa/mm)
Steel	200	.25	58.4	45.5	2110.33
Aluminium	72	.25	59.6	50.0	504.81
PVC	3	.32	62.44	53.3	19.17

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:

$$k_r = \frac{E_r}{(1+v)r} \qquad (2)$$

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

$$2110.33 = \frac{E_r}{(1+0.25)31.5}$$ or E_r=83000 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

Type of bolt	Type of confining pipe, peak load and pull force reduction due to epoxy coating					
	Steel		Aluminium		PVC	
	Peak Load (kN)	Reduction %	Peak Load (kN)	Reduction %	Peak Load (kN)	Reduction %
CT bar Φ28 with Epoxy	110	39	88	15	54	33
CT bar Φ 28 without Epoxy	181		104		80	
Rebar Φ28 with Epoxy	152	5.5	107	14	73	28
Rebar Φ28 without Epoxy	161		125		102	

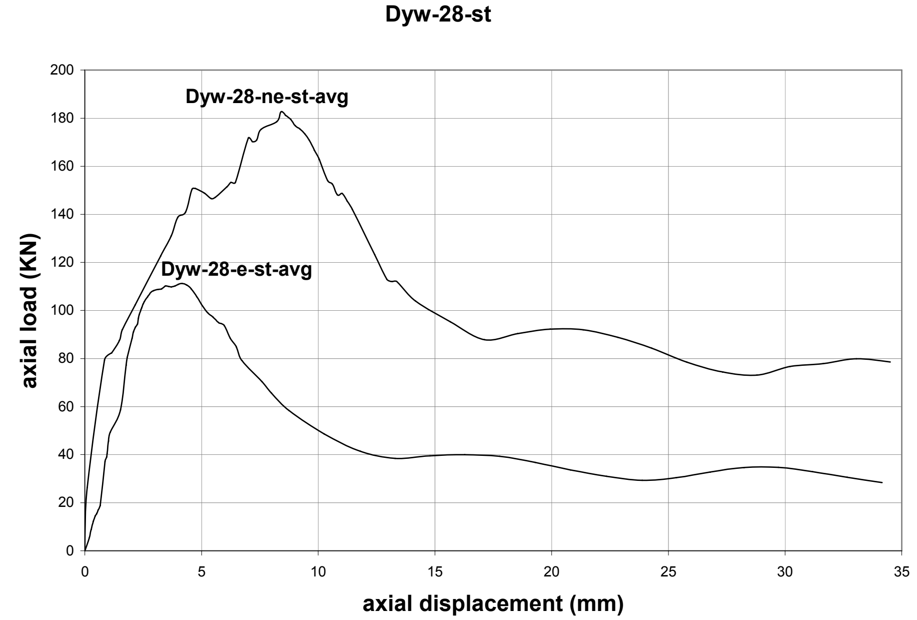

Figure 4 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in a steel pipe.

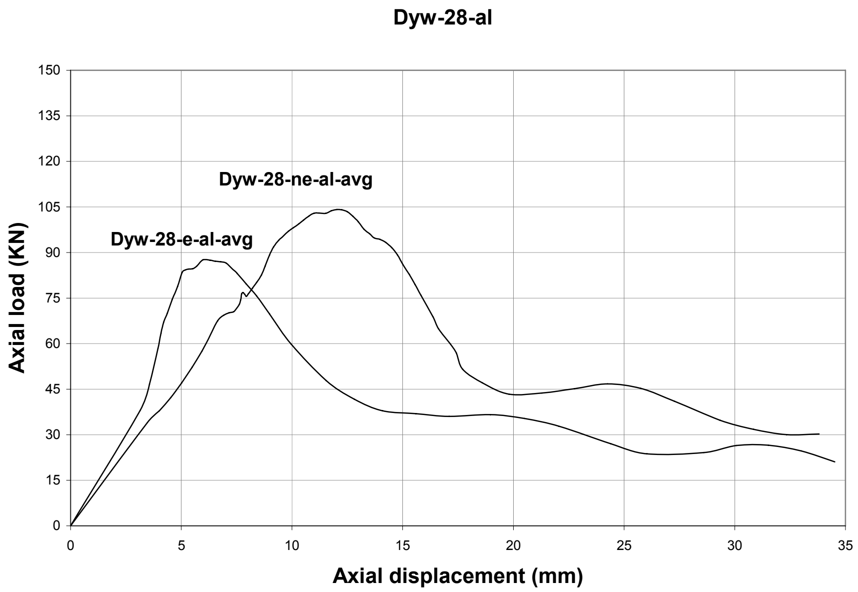

Figure 5 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in an Aluminium pipe.

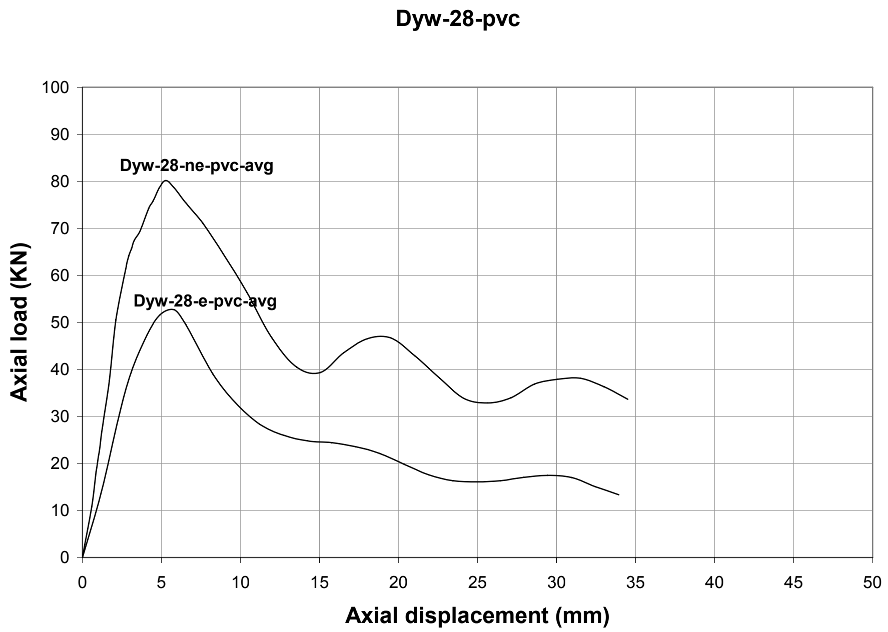

Figure 6 - 28 mm CT bar results for epoxy coated (e) and non epoxy coated (ne) when confined in a PVC pipe

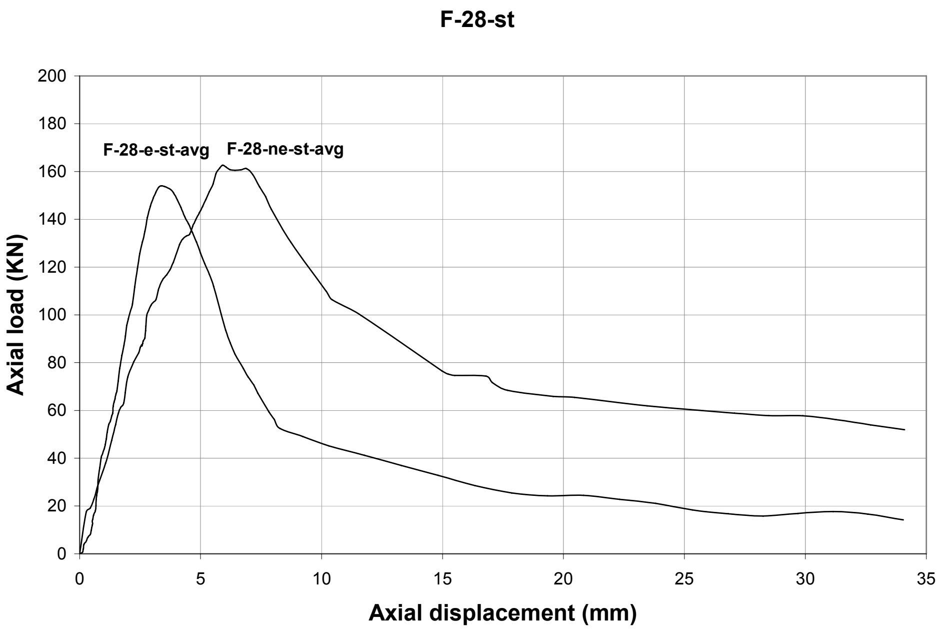

Figure 7 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in a steel pipe

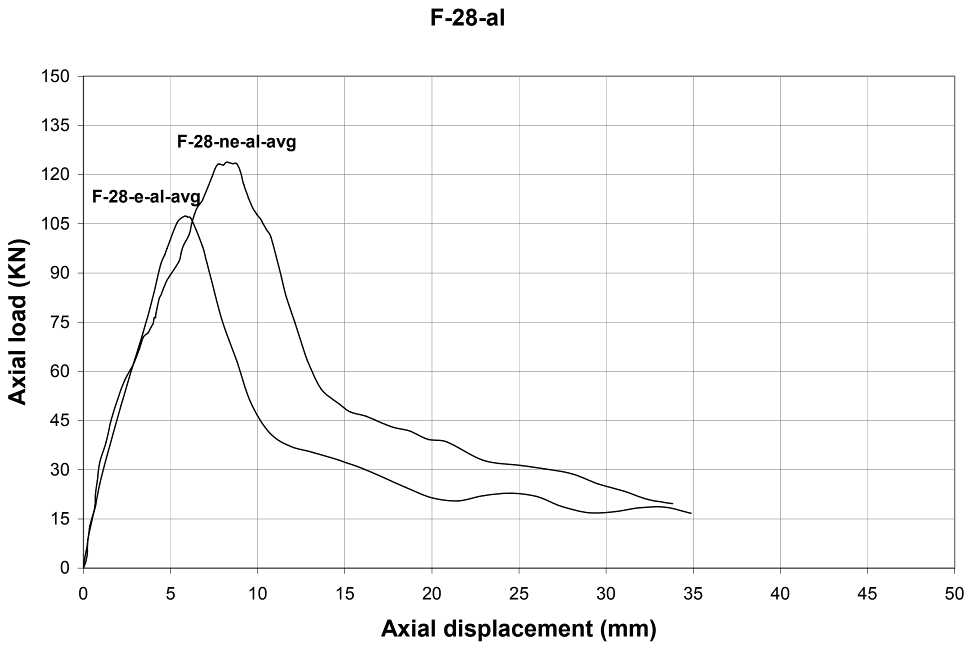

Figure 8 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in aluminium pipe.

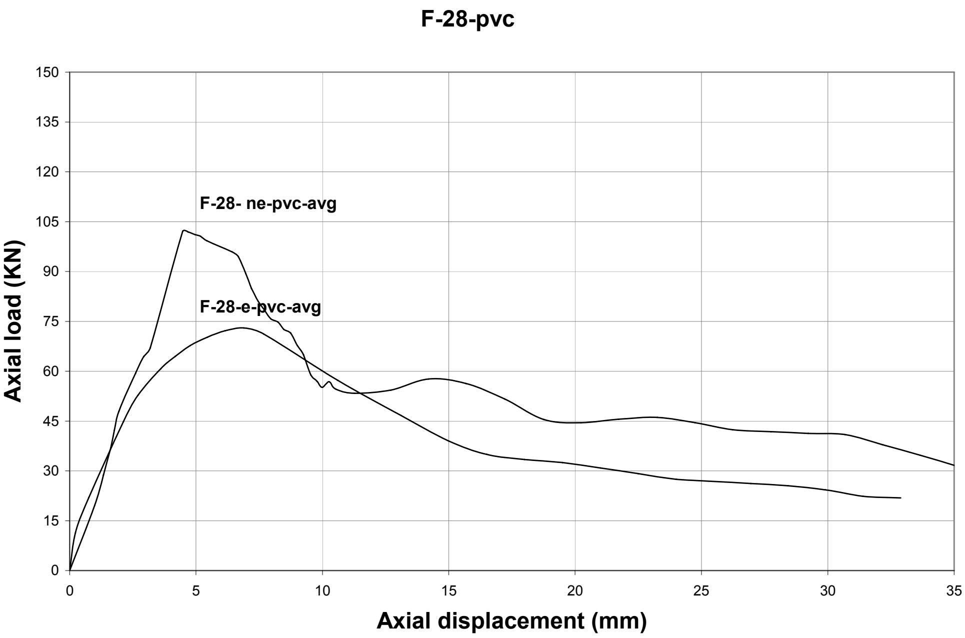

Figure 9 - 28 mm rebar results for epoxy coated (e) and non epoxy coated (ne) when confined in a PVC pipe.

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.

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N OPTIMISATION OF THE BOLT PROFILE CONFIGURATION FOR LOAD TRANSFER ENHANCEMENT

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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, 25.0mm, 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 cementatious 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 cementatious 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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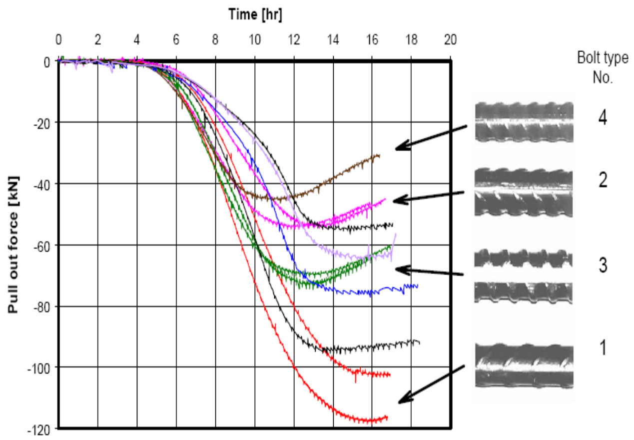


Figure 1 - The load / displacement profiles of different profile spacing bolts. Bolts were installed in a cementacious grout. The rate of loading being at 0.72 mm/hr

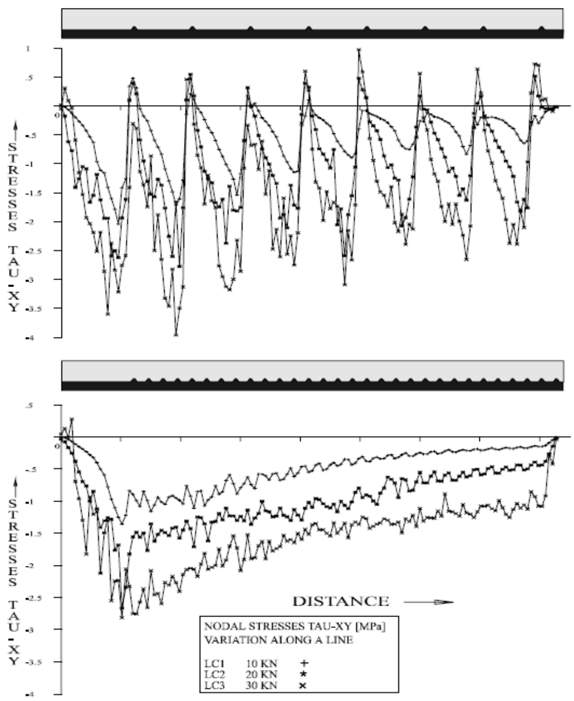


Figure 2 - Axial stress developed on bolts of two different spaced profiles

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

Bolt Type	T1	T2	T3	T2 Bolt Modified		
				G1	G2	G3
Profile Spacing (mm)	12.50	12.50	25.00	25.00	37.50	50.00
Profile Height (mm)	1.00	1.35	1.20	1.35	1.35	1.35
Average Profile Width (mm)	2.25	2.75	3.75	2.75	2.75	2.75
Profile Angle	22.5^{o}	22.5^{o}	22.5^{o}	22.5^{o}	22.5^{o}	22.5^{o}
Bolt Samples						

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 TypeT2, 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 TypeT2. 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)

Bolt Type	Profile spacing (mm)	Average. applied load (kN)	Increase in load with respect to Bolt Type T2 (%)
Bolt Type T2	12.5	215.6	-
Bolt Type T2 G1	25	365.9	69.7
Bolt Type T2-G2	37.5	425.8	97.5
Bolt Type T2-G3	50.0	398.2	84.9

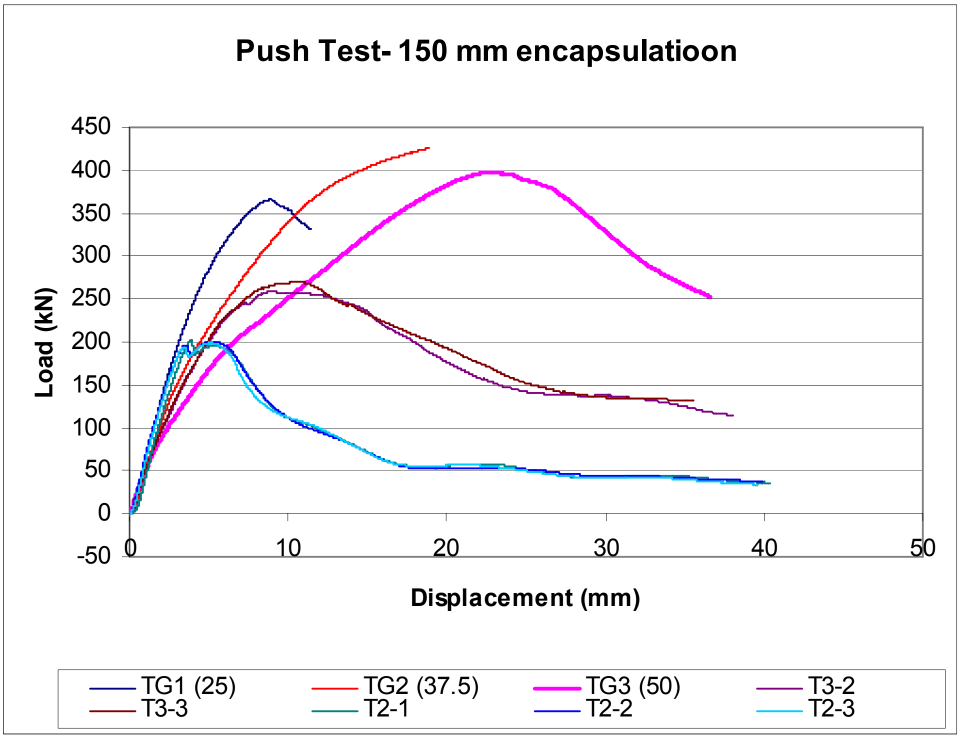


Figure 3 - Push test results of bolts with different profile spacing

Pull Test

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 that 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.

Pull test – 150 mm encapsulation

Push test 150 mm encapsulation

150 mm steel sleeve

Figure 4 - Pull and push testing of bolts with different encapsulation length of 115 and 150 mm

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.

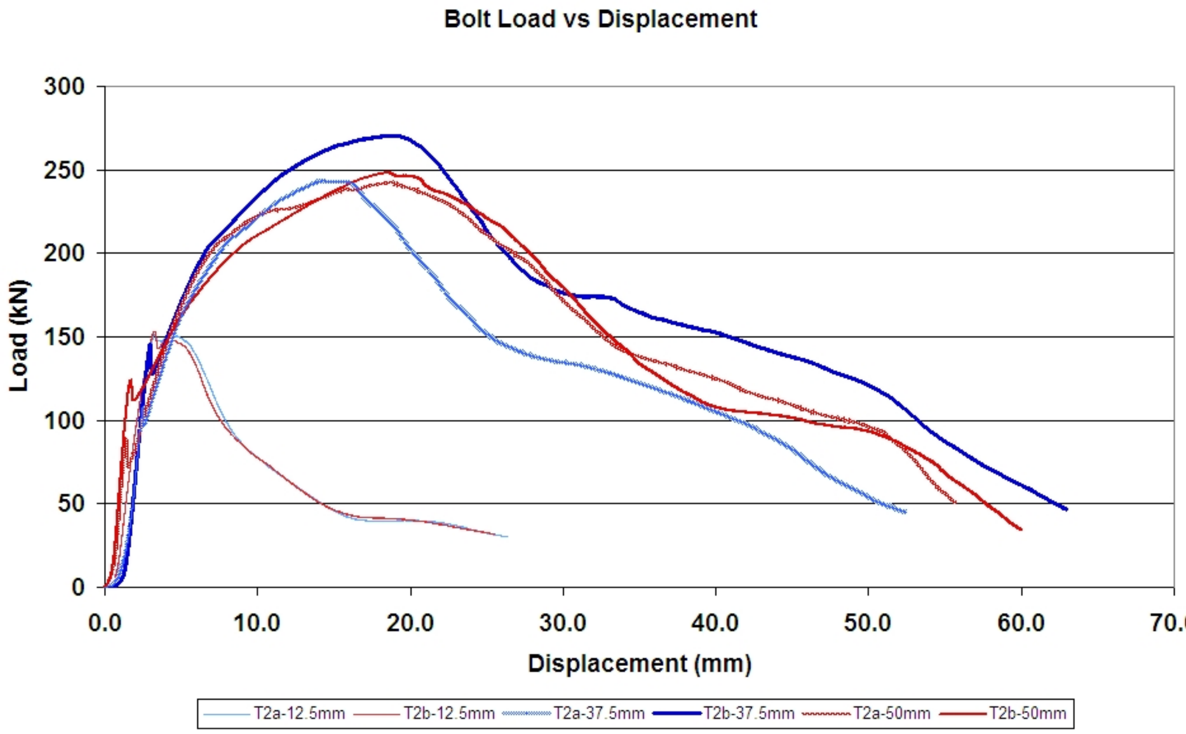


Figure 5 - Load displacement results of different configuration bolts in pull testing

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)

Bolt TypeFigure 3. Fresh air oxygen as measured by tube bundle	Profile Spacing (mm)	Average Pull load (kN)	Change (increase) in load with respect to Bolt Type T2 (%)
Bolt Type T2	12.5	152.23	-
Bolt Type T3 G1	25	215.23	41
Bolt Type T2-G2	37.5	256.55	69
Bolt Type T2-G3	50.0	244.72	61

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 William. 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.

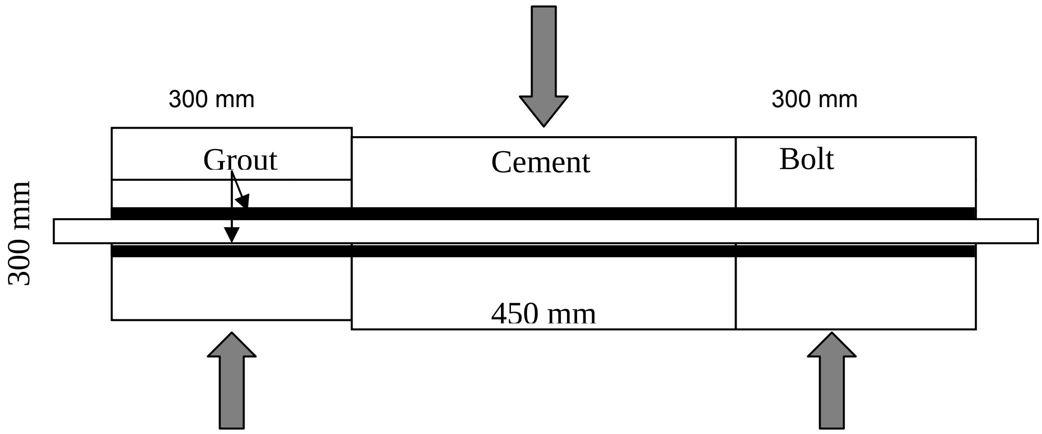
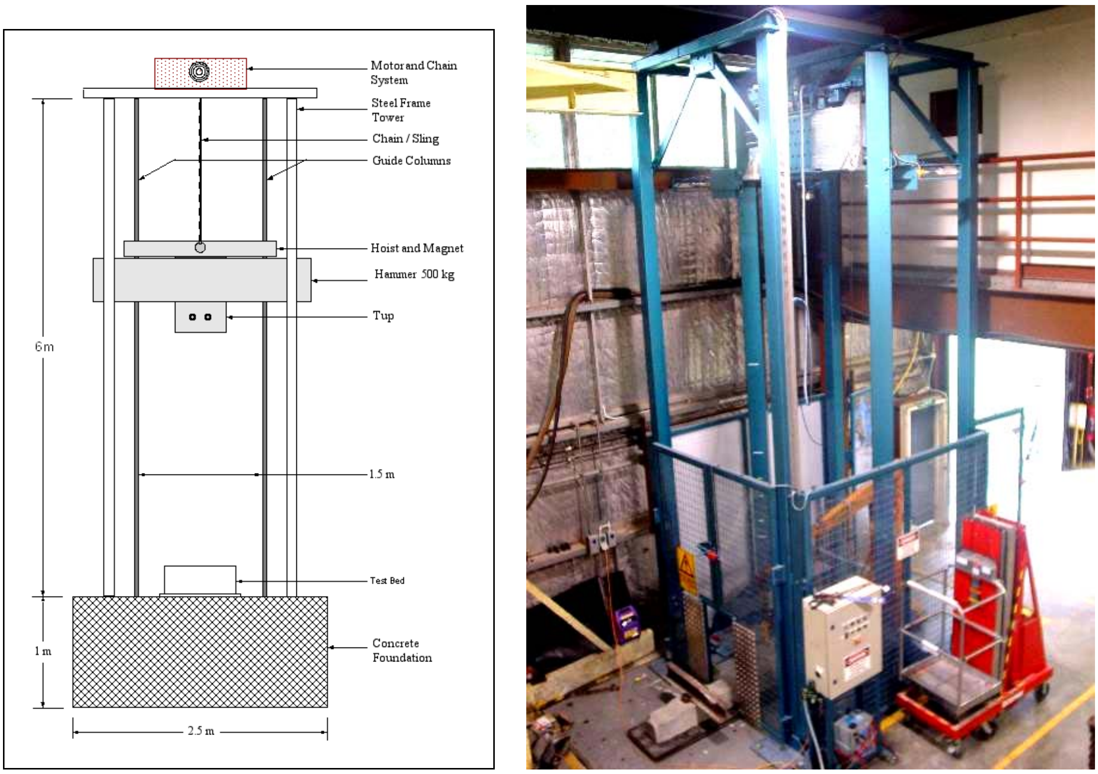


Figure 6 - Large double shear box



Schematic Figure 7 - Large capacity impact load test facility at UoW

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

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

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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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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 qu is the uniaxial compressive strength in MPa, Rn is the Schmidt number and ρ 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:

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

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

$$q_u = 0.094 R_n - 0.383$$

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

$$\ln q_u = 4.3 \times 10^{-2} (R_n \rho_d) + 1.2$$
$$\ln q_u = 1.8 \times 10^{-2} (R_n \rho_d) + 2.9$$

Where ρ_d is the dry density of the rock.

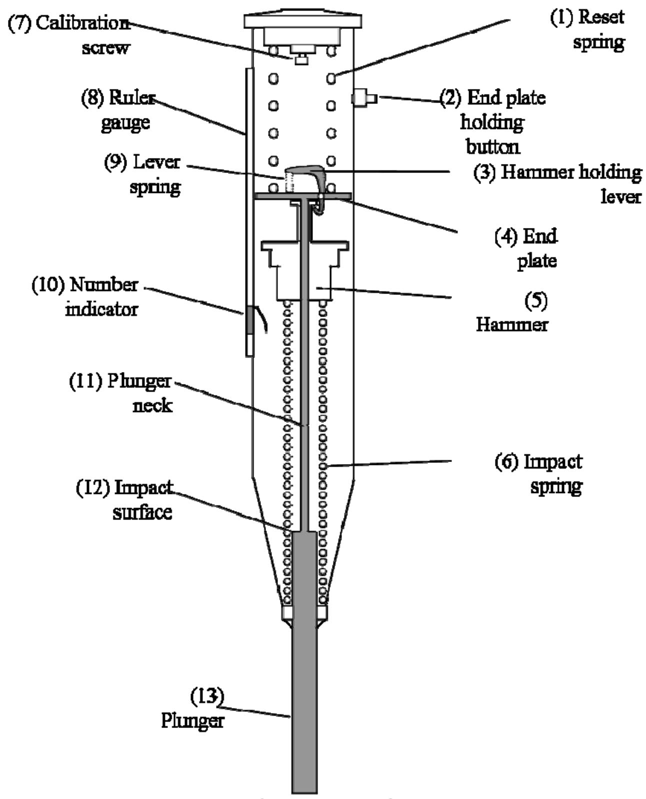


Figure 1 – Details of an L type Schmidt hammer

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.

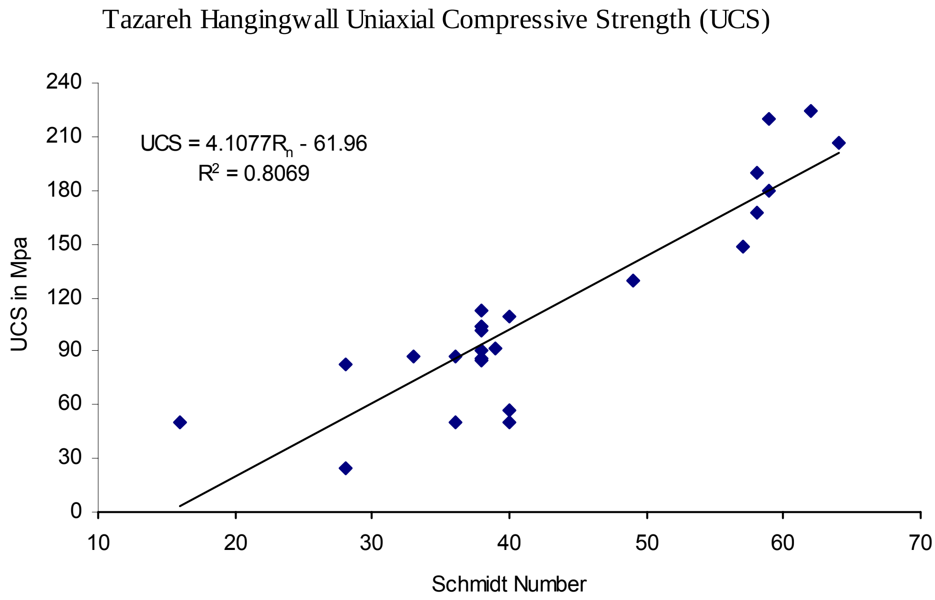
The tests included the application of an L type Schmidt hammer to assess the hardness of the hangingwall rock in as many points as practicable in the stopes as well as in the roadways. In each point an about 20cm by 20cm surface of the rock was prepared by peeling the remaining coal and cleaning the 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.



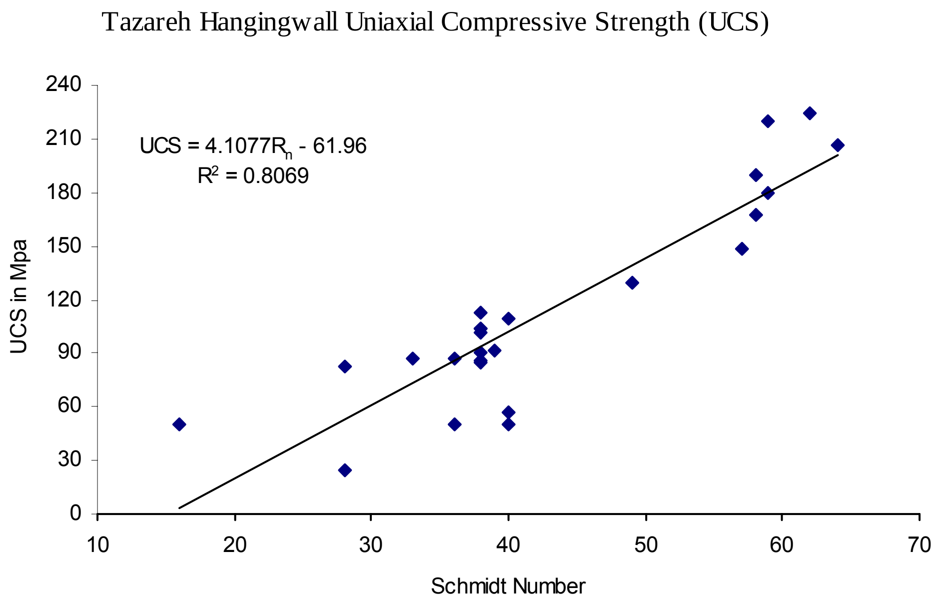


Figure 2 - Relationship between Schmidt number and UCS for the roof rock of Tazareh colliery

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

$$UCS = 4.1077Rn - 61.96 \qquad (1)$$

The correlation coefficient of the relationship, R2, 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[1] and Rao Balusu[1]

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.

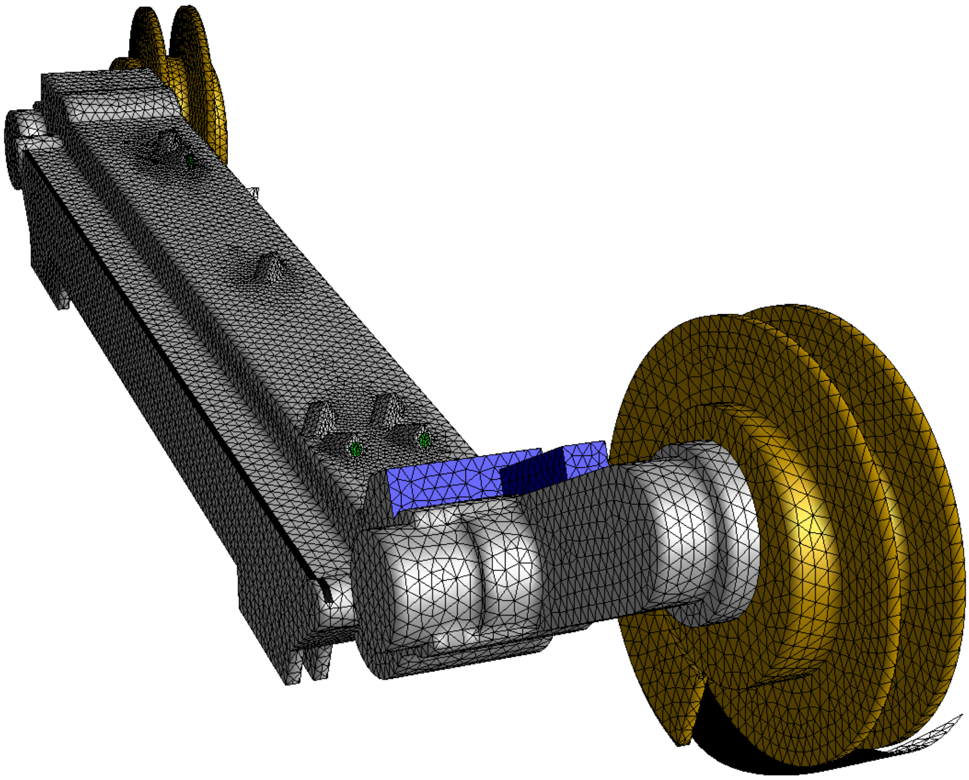


Figure1 - Computational mesh of the CFD model – the longwall shearer

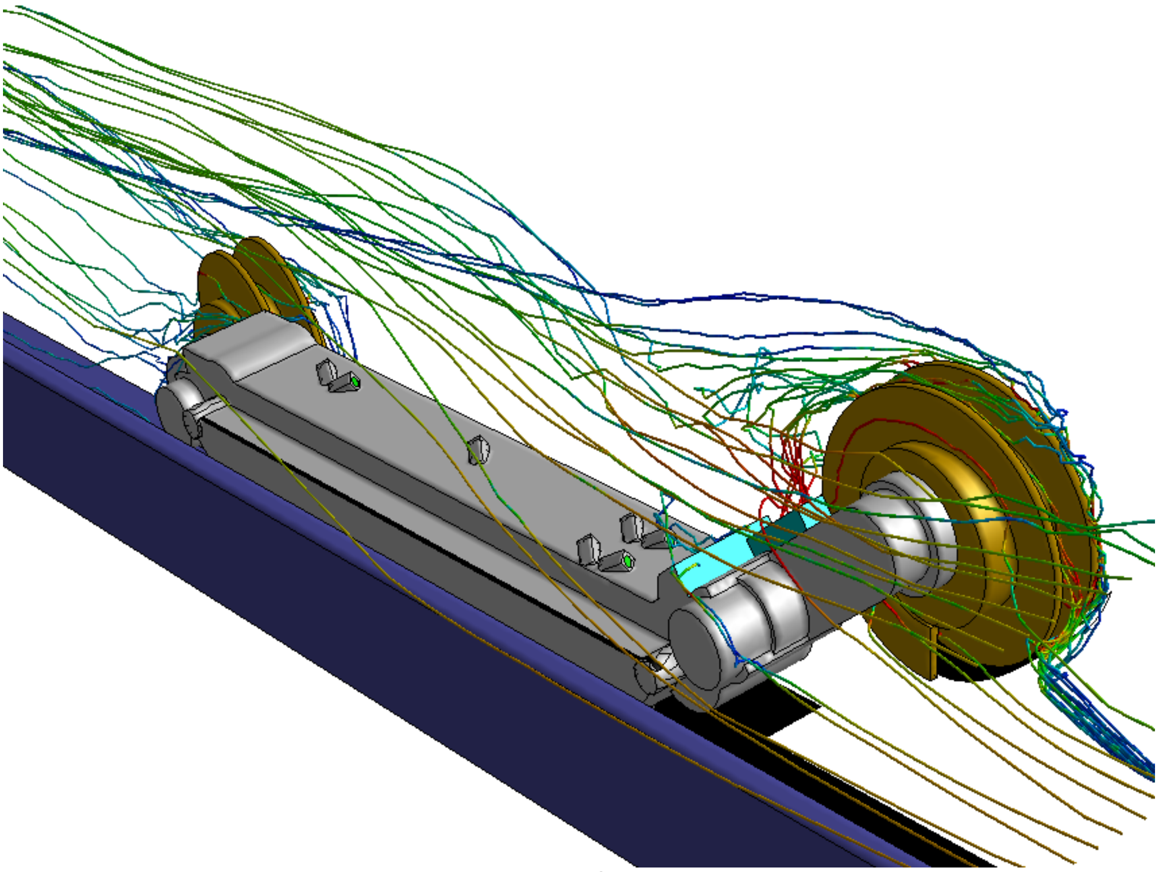


Figure 2 - Respirable dust particle tracking in CFD modelling around the longwall shearer

CFD MODELLING OF VENTILATION AND DRUM CUTTING SEQUENCE

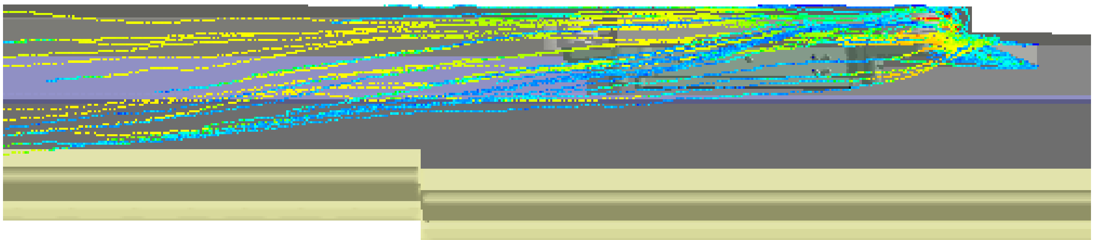
CFD results indicate that the dispersion of respirable dust on a longwall face is largely dependent on the location of dust generation and airflow patterns in that area. The dust dispersion from the cutting drum and face spalling is slightly more extensive on thick seam longwall faces with low airflow rate of around 35 m^3/s compared with higher airflows 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.

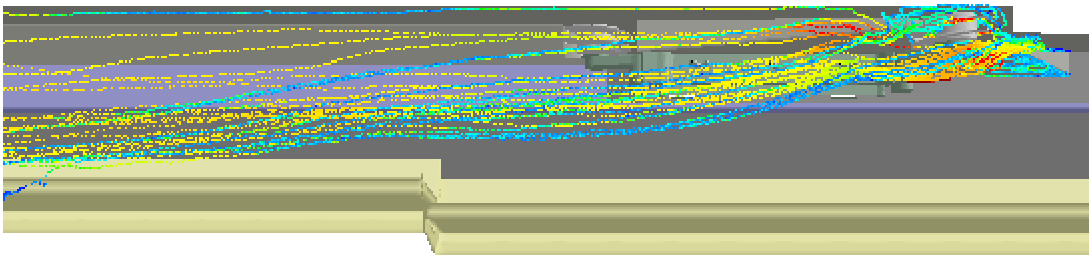
CFD MODELLING OF SPRAYS/VENTURI'S AND SHEARER CLEARER SYSTEM

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.

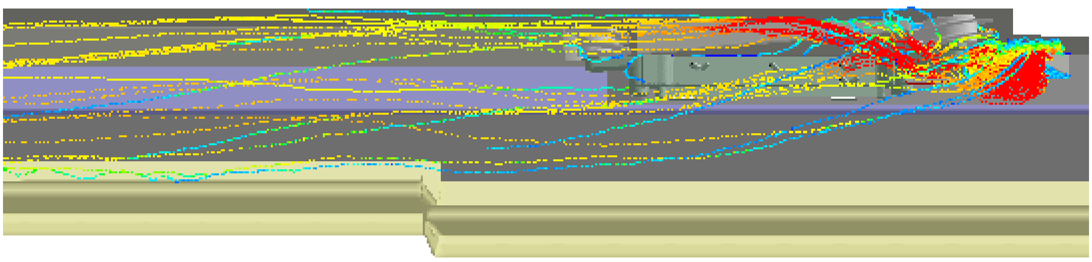
As shown in Figure 3, a Shearer Clearer operating at 0.25 m^3/s flow rate would help reduce the dispersion of dust particles towards the face operators by inducing the dust cloud towards the face area, and at a higher flow rate of 0.5m^3/s, the performance of the Shearer Clearer system can be improved 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.



(a) Base case



(b) Shearer clearer induced airflow – 0.25 m^3/s



(c) Shearer clearer induced airflow – 0.5 m^3/s

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 shear 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

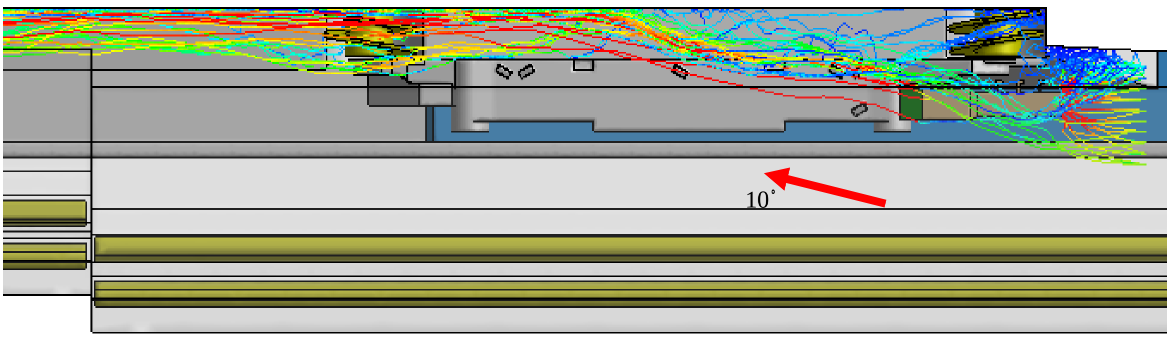
To prevent dust from becoming airborne, one of the several successful control measures in longwall faces is the use of dust scrubbers which cleanse the air and help to prevent the contaminated air from reaching areas used by mine personnel. Scrubbers have proven to be highly successful in reducing airborne 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 $8m^3/s$.

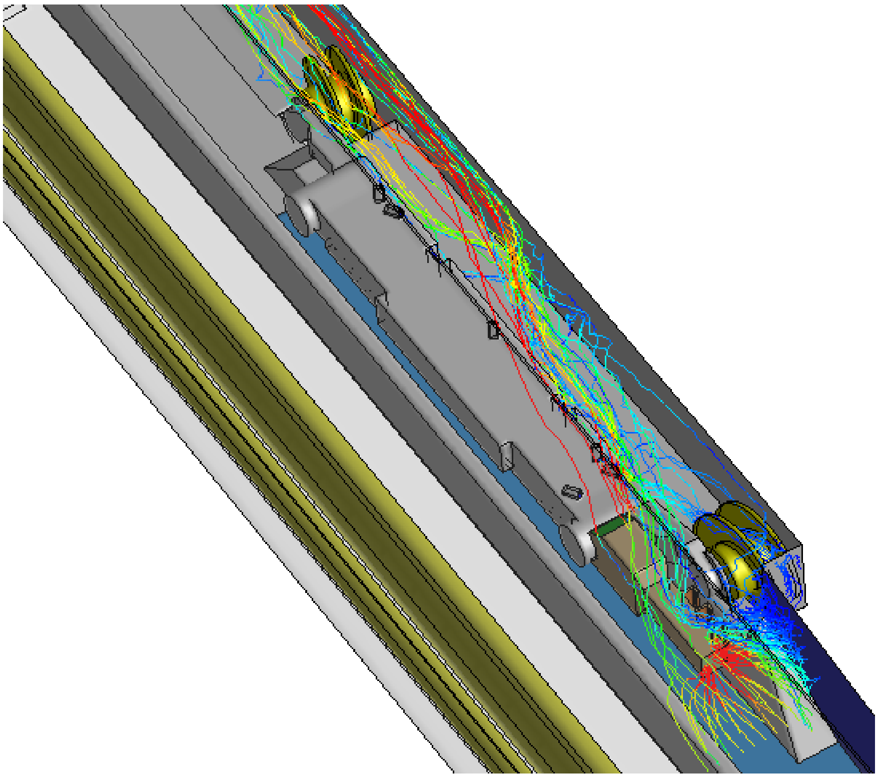
The scrubber capacity must be appropriate in relation to the prevailing face airflow rates (velocities) around the shearer. Scrubbers with a capacity around $8m^3/s$ would have a good impact on dust capture and dust flow patterns at moderate face ventilation of around 55 m^3/s. However, the capture efficiency 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.

ACKNOWLEDGEMENTS

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

Stewart Gillies[1] and Hsin Wei Wu[1]

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 region and so has many advantages over instruments that measure from a fixed-point location. It can quickly 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, manning 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.

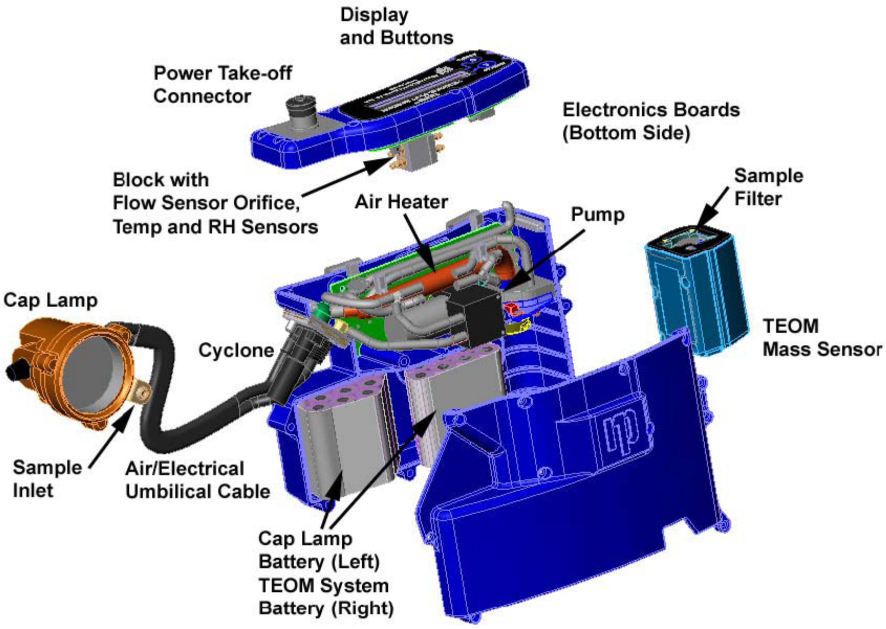

Figure 1 - Major components of the PDM

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 that 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.

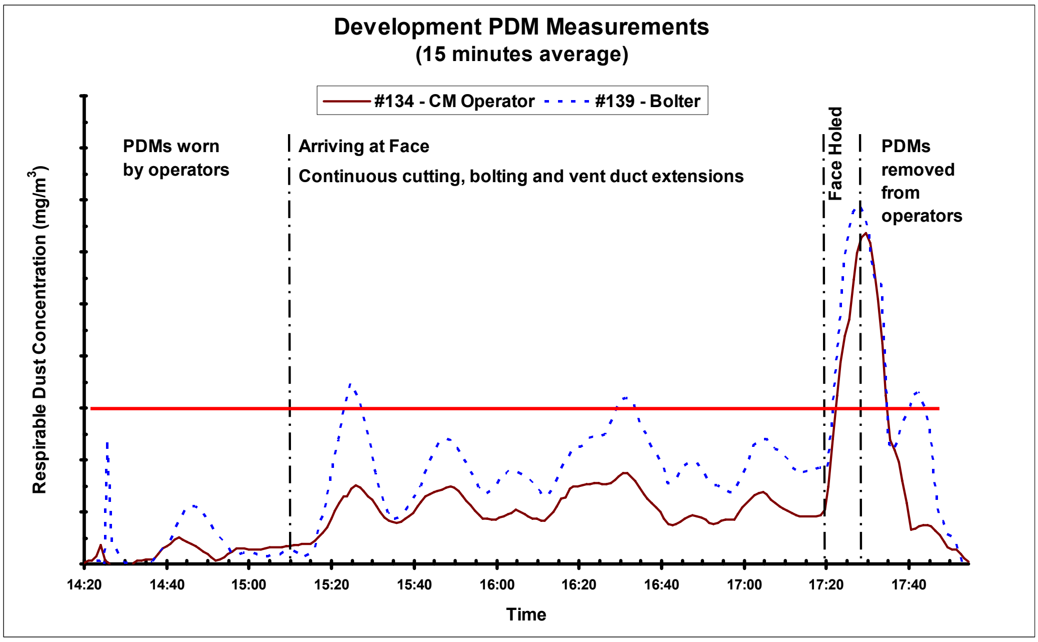

Figure 2 - Development Face PDM results

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.

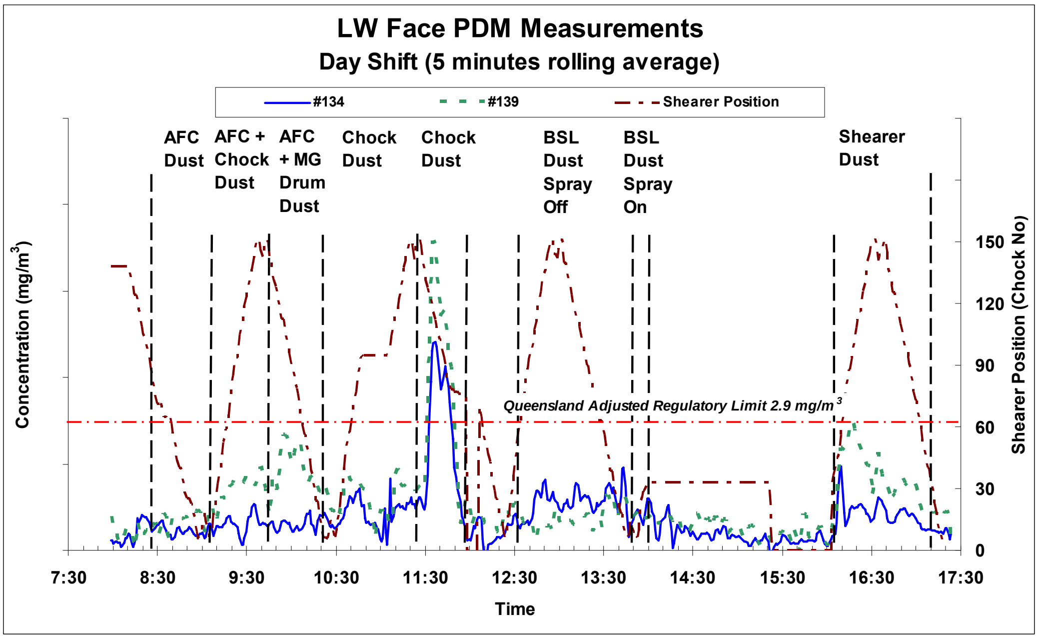

Figure 3 - LW Face Dust Surveys Shearer Position and dust monitored points and Levels

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.

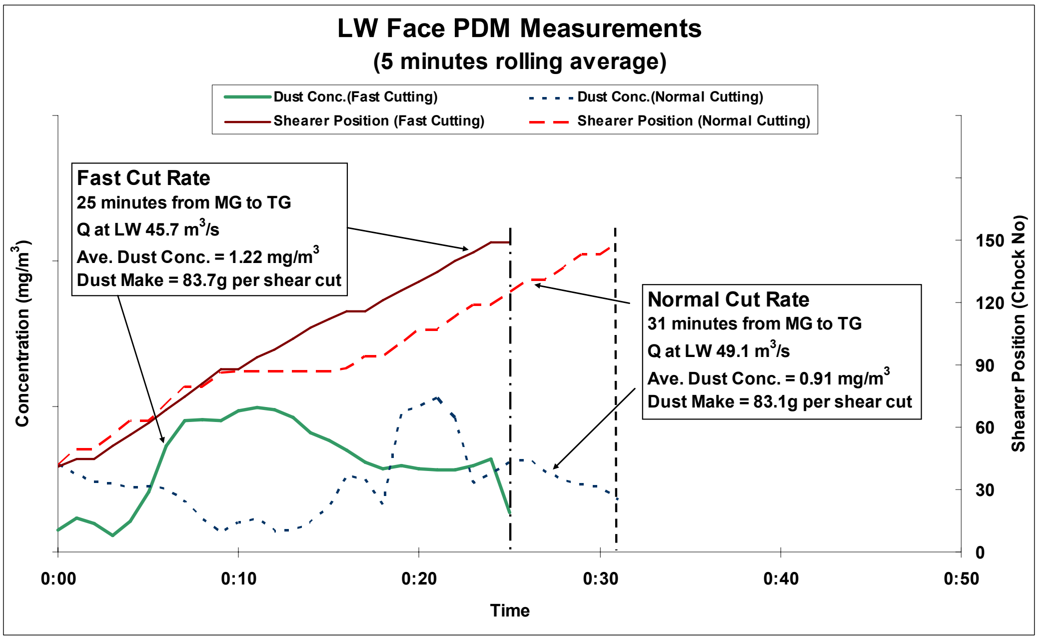

Figure 4 - Variation of dust make with shearer advance rates

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.

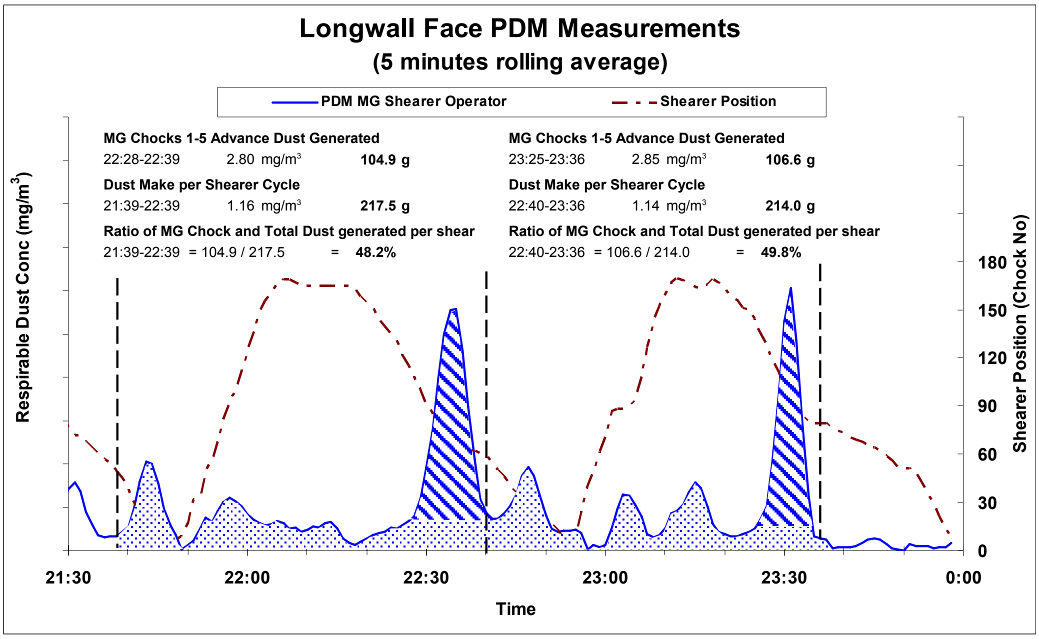

Figure 5 - **MG chocks 1-5 advance dust compared with total LW face dust at shearer MG operator position**

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/m^3 submicron particulate matter, 0.16 mg/m^3 total carbon particulate and 0.1 mg/m^3 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/m^3 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 straight forward 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 m^3/s ventilation was measured in the MG and about 35 m^3/s in the TG. There was a raise borehole upcasting some air.

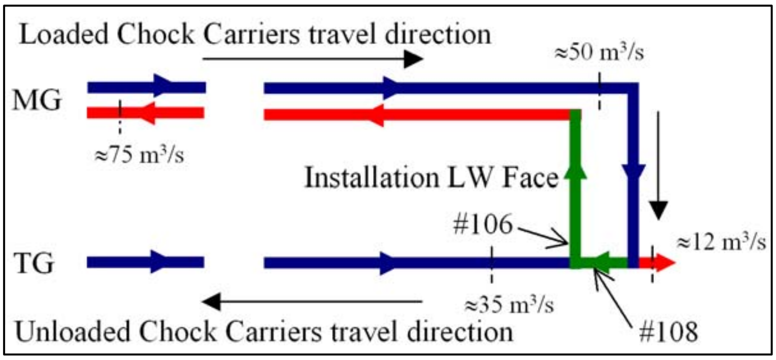

Figure 6 - Longwall ventilation-chock carriers travel in on MG and out on TG

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.

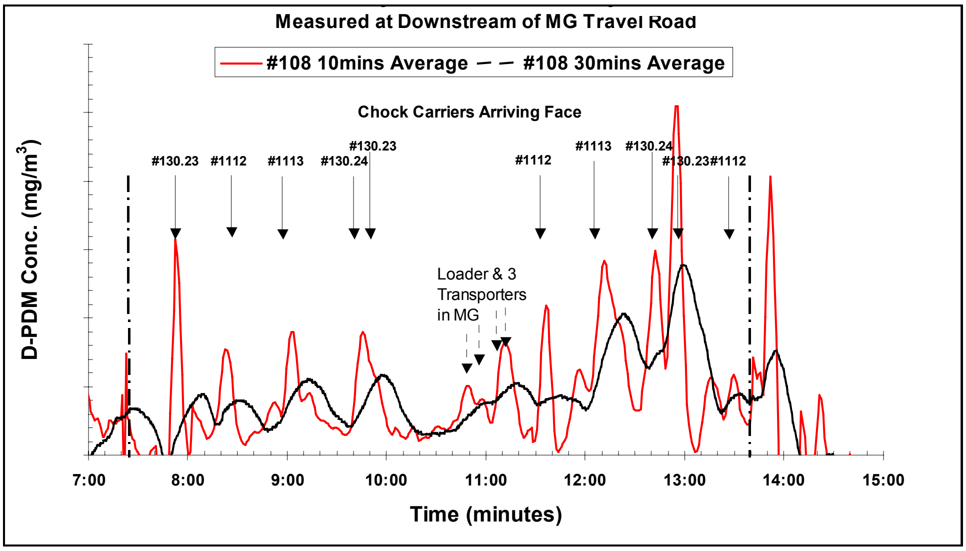

Figure 7 - Observations on results at monitor 108 fixed location

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 m^3/s ventilation was measured in the MG and about 39 m^3/s in the TG. There was a back borehole downcasting about 11 m^3/s. Three chock carriers were available and a total of four chocks were moved.

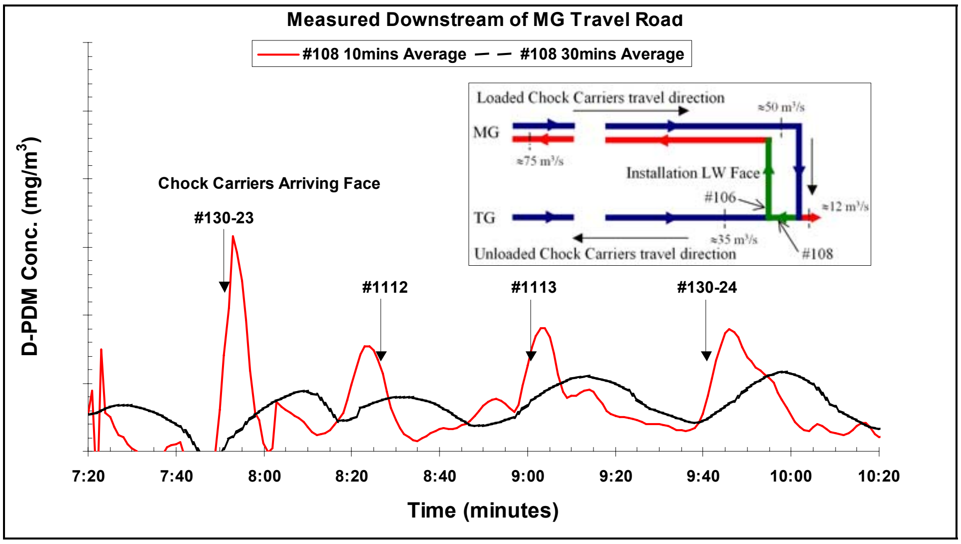

Figure 8 - Observations on results over a three hour period at monitor 108 fixed location

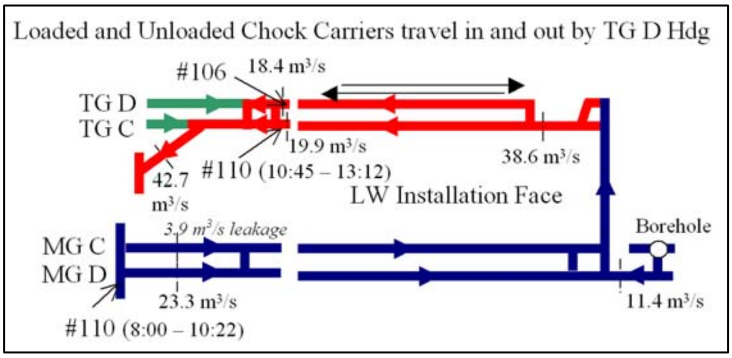

Figure 9 - Longwall ventilation-chock carriers travel in and out on TG

Figure 10 shows readings from fixed location monitoring of chock movements in the face area and nearby, D-PDM monitors #110 and from monitoring all air that had passed through the longwall panel, D-PDM monitors #106. The trace of monitor #110 illustrates clearly the arrival and departure of individual chock carriers at the Face TG end and subsequent movement chock repositioning by a diesel "shunting mule", Eimco 936 1123. The trace of monitor #110 illustrates the additional DPM in the return air picked up from the travel of chock carriers along the length of the TG roadway. Both traces register the activity although from different air sources and it can be seen that as traffic became heavier the level of DPM increased and when the traffic eased off the level of DPM reduced.

Mine 3 results were analysed to identify sources and levels of DPM within the panel by strategically placing the real time DPM monitors within the longwall panel as shown in Table 1. The DPM sources (μg/s) in the table are calculated by knowing the air quantity (m^3/s) and the DPM concentration ($\mu g/m^3$) at various locations within the panel ventilation circuit. There were significant DPM levels in MG Heading D due to outbye traffic and in particular the passage of chock carriers in the Mains intake air stream as they passed to the panel TG. There were also 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.

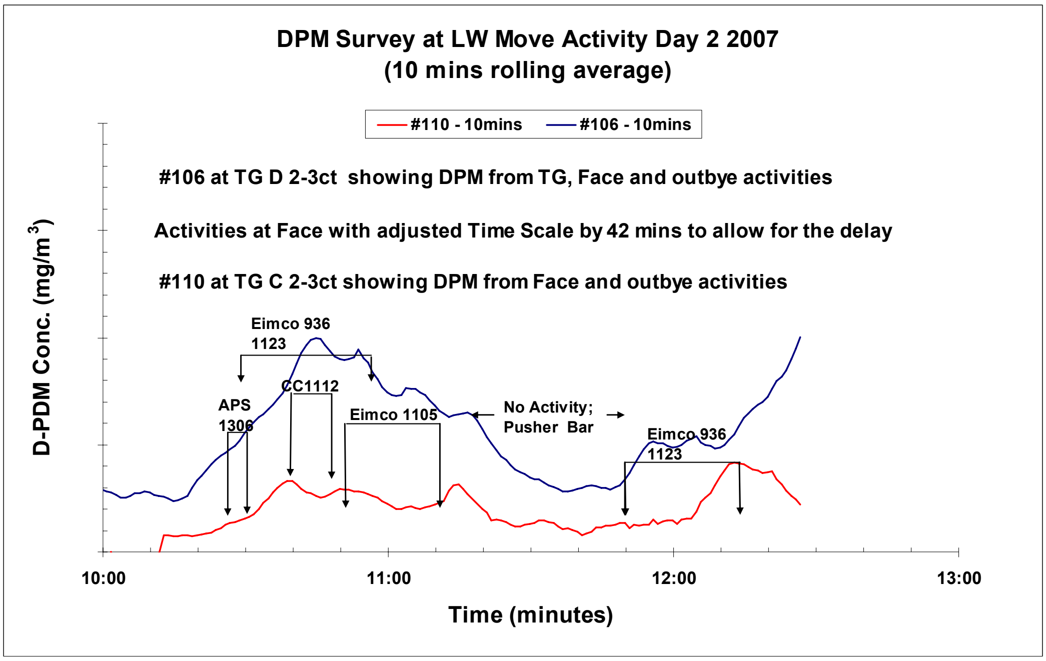

Figure 10 - DPM make from LW face activity, #110 compared with DPM make from both face and TG transport activities, #106

Table 1 - Sources of DPM identified in the installation LW panel

Location	Sources, µg/s	%	Comments
MG, C & D Hdgs	3.03	18.6	Mains air at MG panel entrance
Borehole	0.00	0.0	Situated at the back of LW panel, fresh air
LW Face	4.77	29.2	Shunting Mule or LHDs
TG D Hdg	6.96	42.6	Chock carriers travel way
TG C Hdg	0.00	0.0	No diesel activity
Leakages	1.57	9.6	Mains air; coffin seal & double doors
Measured Total	**16.32**	**100.0**	

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

Time	Location	In/Out	Distance m	Time mins	Speed, m/s	Air Vel m/s	Air Travel Time mins
Chock Carrier APS 1306							
9:53	TG26 2ct	In	3,400	34	1.66	1.29	43.9
10:27	Face						
	Machine Against Air			**Machine/Air Rel Velocity, m/s = 2.95**			
10:31	Face	Out	3,400	26	2.18	1.29	43.9
10:57	TG26 2ct						
	Machine With Air			**Machine/Air Rel Velocity, m/s = 0.89**			
Chock Carrier CC 1112							
10:12	TG26 2ct	In	3,250	28	1.93	1.29	41.9
10:04	TG26 36ct						
	Machine Against Air			**Machine/Air Rel Velocity, m/s = 3.22**			
10:05	TG26 36ct	Out	3,250	17	3.18	1.29	41.9
11:07	TG26 2ct						
	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 rime 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 m^3/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/m^3 in a particular airway
- Air way ventilation quantity at that point is measured in m^3/s
- DPM "make" is the product of the two i.e. mg/m^3 x m^3/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 (m^3/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/m^3) 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

Vehicle	Engine kW	Av Make Av/Max* mg/s	U/G Test 1 mg/s	U/G Test 2 mg/s	U/G Test 3 mg/s	U/G Test 4 mg/s
Toyota	55	3.05/9.21	0.08, idle	-	-	-
SMV Drifty	63	2.97/5.94	2.14, idle	1.34, idle	2.59	-
Eimco, CAT 3306	Av 105	3.14/9.81	1.5, idle	9.02, rev	3.27	2.07

**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.

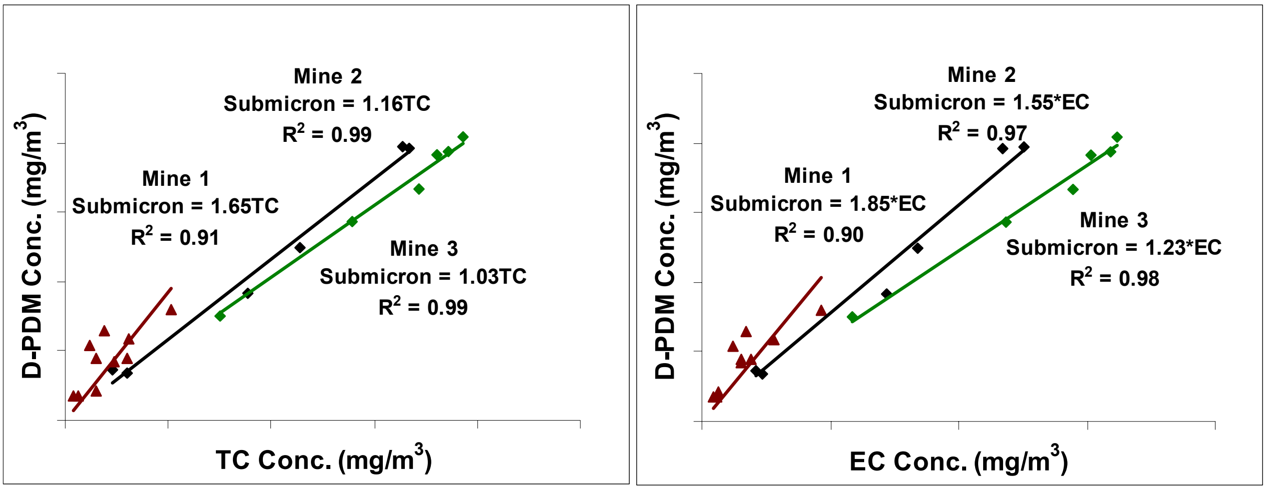

Figure 11 - Mine individual relationships between TC or EC and Submicron DPM results.

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.

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MOISTURE CONTENT IMPACT ON THE SELF-HEATING RATE OF A HIGHLY REACTIVE SUB-BITUMINOUS COAL

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ABSTRACT: A New Zealand power station has been importing subbituminous coal, which has oocasionally 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 sub-bituminous 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 sub-bituminous 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

	INDO1A	*INDO2A - E*
Moisture (%, ad)	17.7	17.7
Moisture (%, ar)	25.2	24.3
Ash Content (%, db)	2.0	1.8
Volatile Matter (%, dmmf)	51.6	51.6
Calorific Value (Btu/lb, mmmf)	9755	9755

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

Test Sample	*As Received Moisture (%)*	*Test Moisture Content (%)*
INDO2A	24.3	0.0
INDO2D	24.3	7.3
INDO2C	24.3	12.5
INDO2B	24.3	16.7
INDO2E	24.3	24.0

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

Test Sample	*Test Moisture Content (%)*	*Self-Heating Rate (°C/h)*
INDO2A	0.0	12.92
INDO2D	7.3	21.62
INDO2C	12.5	17.07
INDO2B	16.7	16.07
INDO2E	24.0	10.87

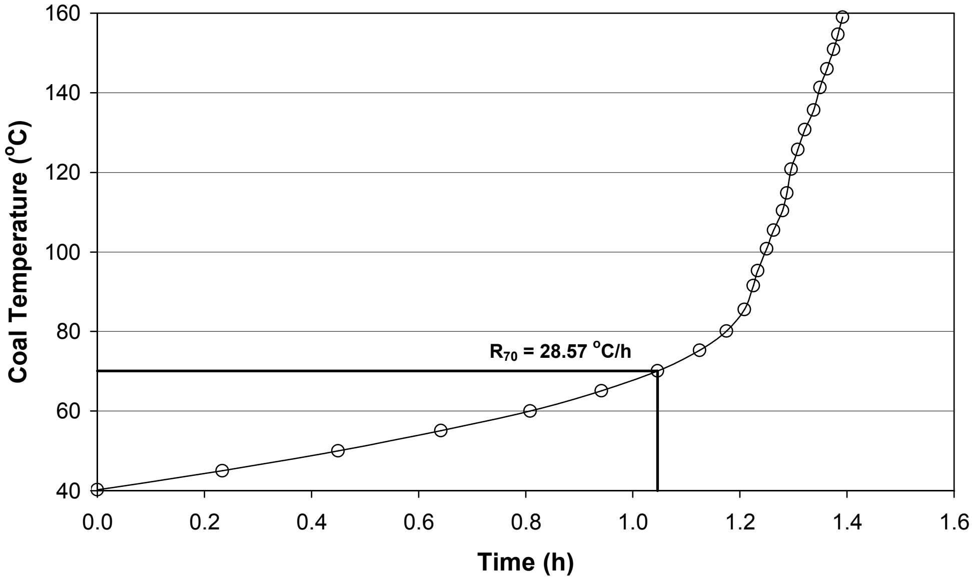

Figure 1 - R_{70} self-heating rate index for INDO1A sample

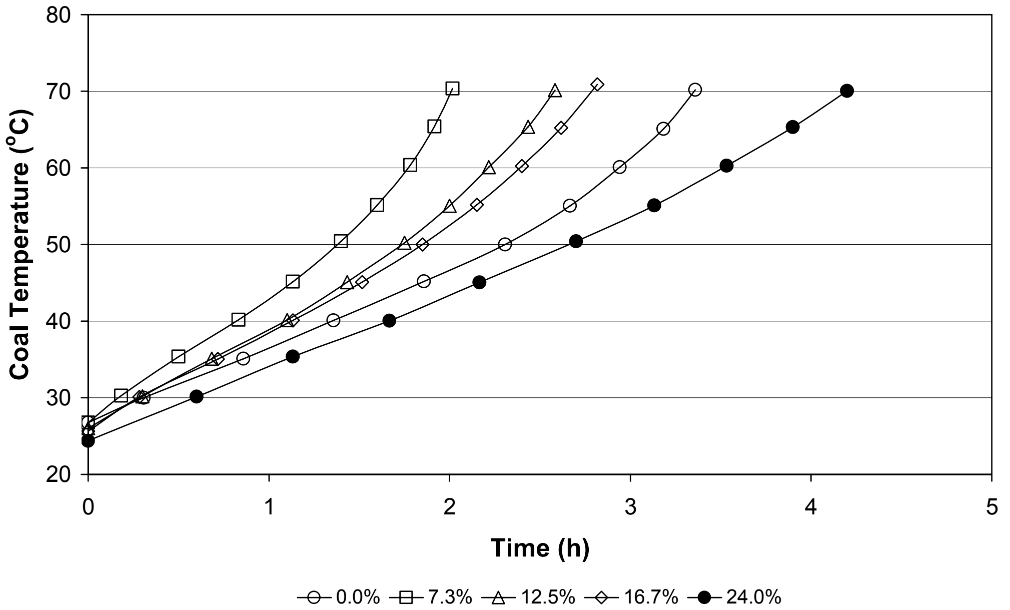

Figure 2 - Initial self-heating rate curves for the INDO2A-E replicates at different moisture content

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 REPLICATES

Test Sample	*Test Moisture Content (%)*	*Time to Thermal Runaway (h)*
INDO2A	0.0	3.86
INDO2D	7.3	5.22
INDO2C	12.5	8.65
INDO2B	16.7	12.15
INDO2E	24.0	23.20

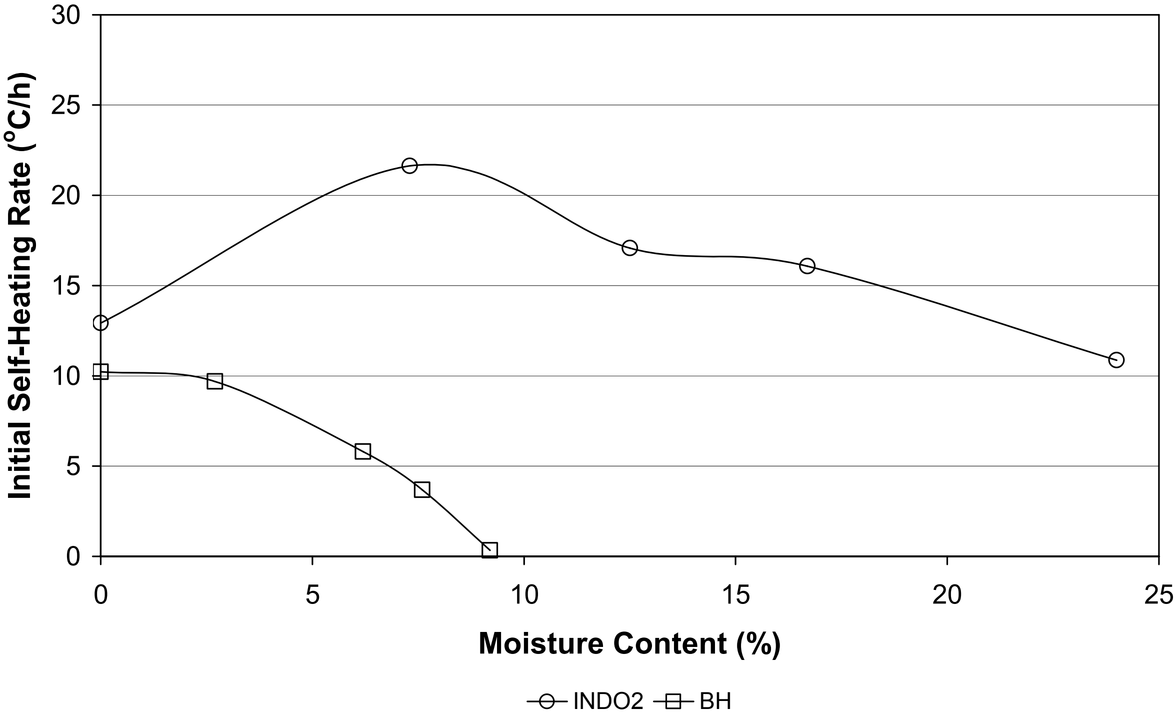

Figure 3 - Comparison between initial self-heating rates and moisture contents of INDO2 replicates and Boundary Hill coal

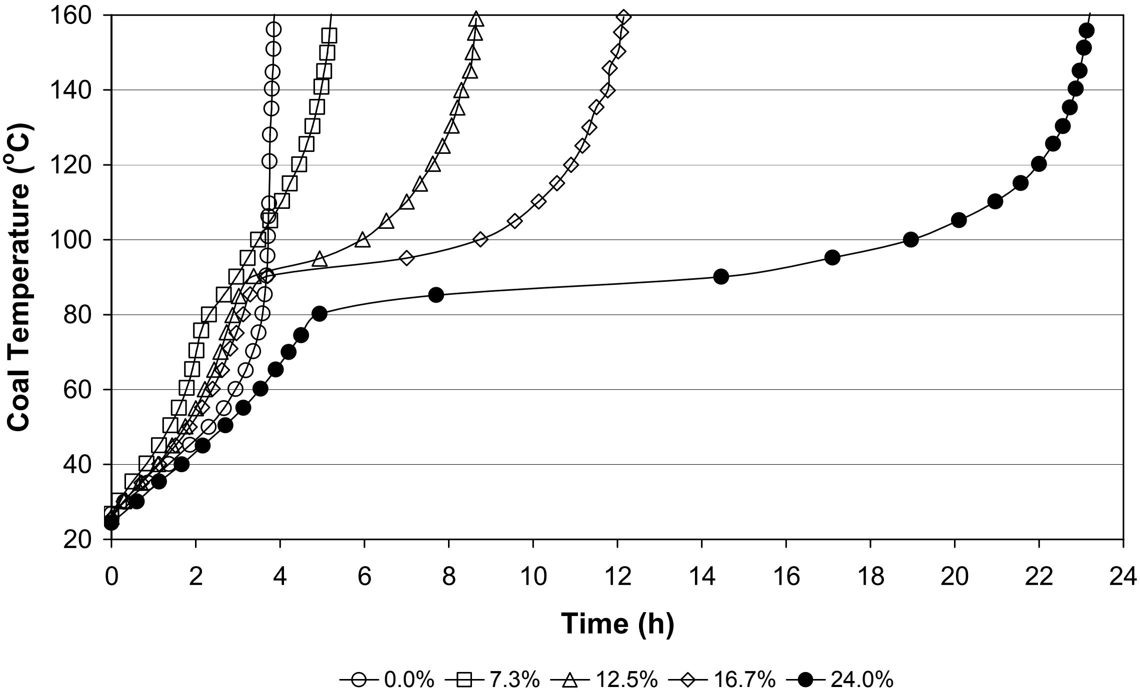

Figure 4 - Complete self-heating curves for the INDO2A-E replicates at different moisture content

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.

ACKNOWELDGEMENTS

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DEVELOPMENT OF A SITE SPECIFIC SELF-HEATING RATE PREDICTION EQUATION FOR A HIGH VOLATILE BITUMINOUS COAL

B Basil Beamish[1] and Wade Sainsbury[1]

ABSTRACT: The R_{70} 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 R_{70} and ash content, with the exception of one anomalously low result. An ash analysis of the sample showed that it had a high sodium (Na_2O) content in response to the presence of the sodium zeolite mineral, analcime. A multiple regression of the R_{70}, ash content and sodium (Na_2O) 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 R_{70} 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, R_{70} 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 R_{70} coal self-heating rate relationship with rank is non-linear. Beamish and Blazak (2005) also showed that R_{70} 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 R_{70} 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 R_{70} test procedure (Beamish *et al.*, 2005; Beamish and Clarkson, 2006). However, this is the first time that the R_{70} 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,

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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 anlaysed using X-Ray Diffraction (XRD) and the results obtained showed the presence of the sodium zeolite mineral, analcime ($NaAlSi_2O_6 \cdot H_2O$). 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.

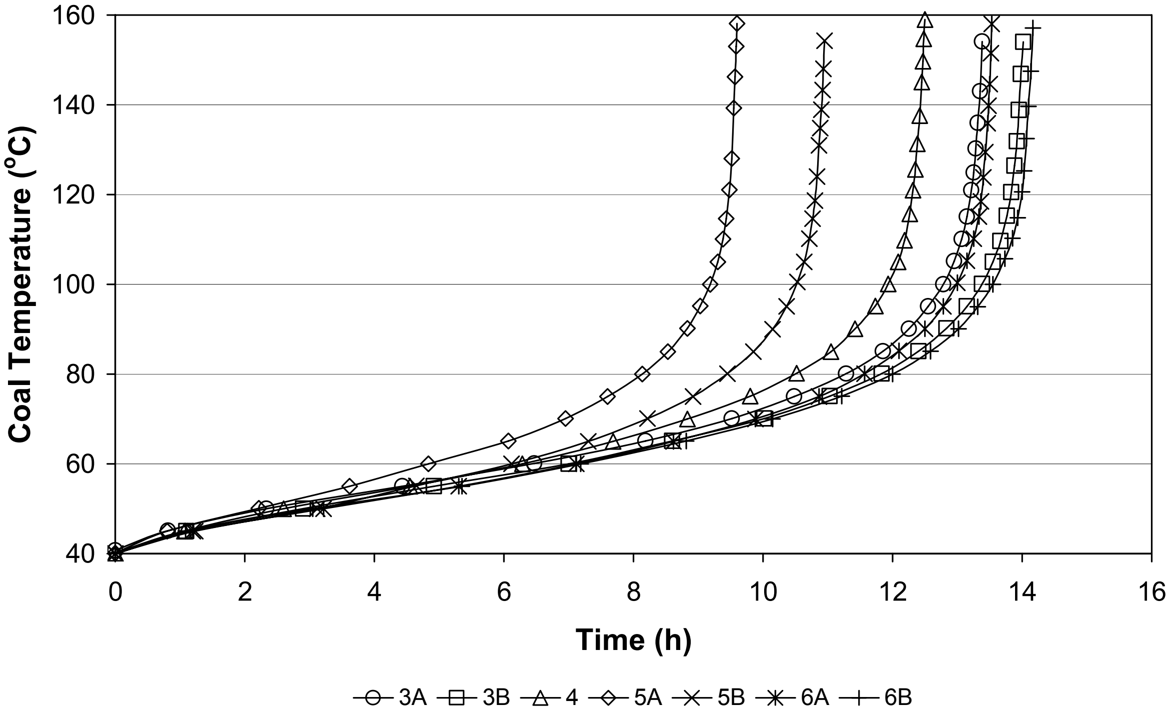

Figure 1 - Adiabatic self-heating curves for a high volatile bituminous coal

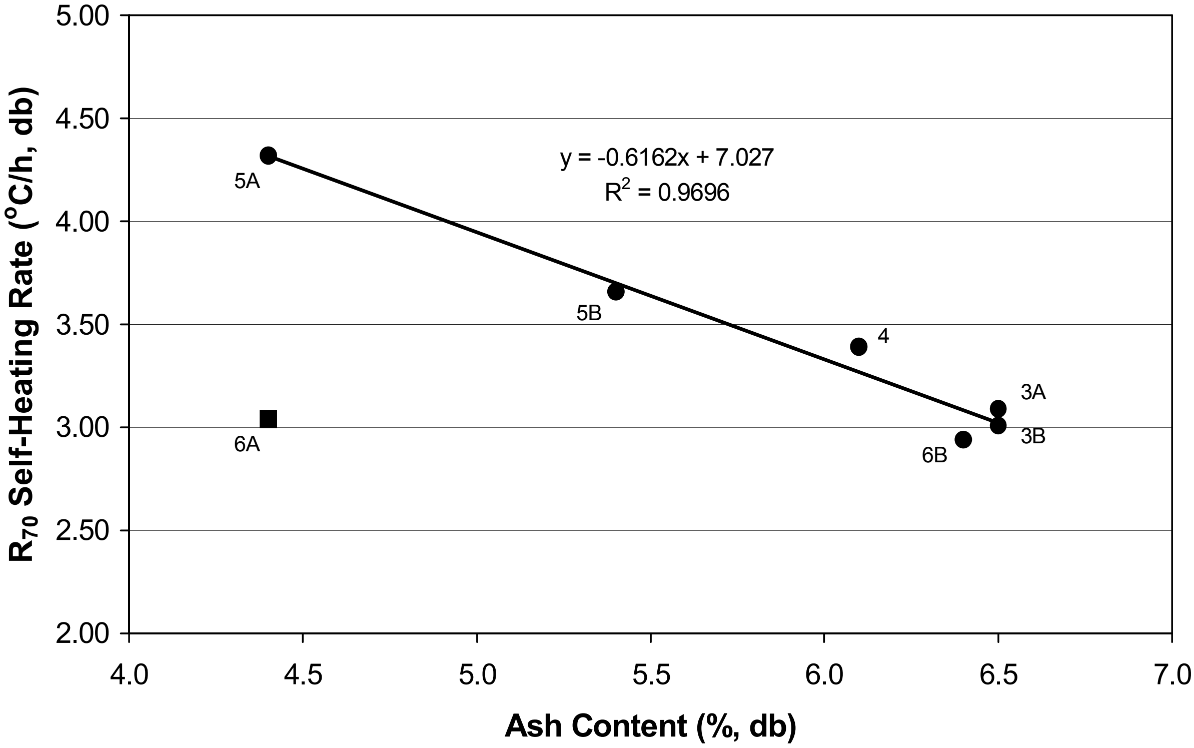

Figure 2 - Relationship between R_{70} self-heating rate and ash content

Table 1 - Ash Analysis Results

Ash Composition (%, db)	3A	3B	4	5A	5B	6A	6B
Silicon (SiO_2)	56.0	56.7	70.0	49.0	53.0	50.5	54.5
Aluminium (Al_2O_3)	30.2	31.0	18.1	28.0	27.0	25.5	23.5
Iron (Fe_2O_3)	4.6	4.8	4.7	8.3	5.4	5.2	4.0
Calcium (CaO)	0.8	0.82	1.2	4.5	4.1	4.0	4.9
Magnesium (MgO)	1.2	1.2	1.0	1.9	1.5	1.3	1.3
Sodium (Na_2O)	2.7	3.0	1.6	3.4	2.5	10.0	3.5
Potassium (K_2O)	0.47	0.48	0.41	0.47	0.63	0.44	0.38
Titanium (TiO_2)	1.5	1.7	1.7	1.4	1.4	1.4	1.7
Manganese (Mn_3O_4)	0.02	0.01	0.02	0.03	0.01	0.01	0.01
Phosphorous (P_2O_5)	0.14	0.15	0.12	0.56	1.50	1.10	1.00
Sulphur (SO_3)	0.61	0.43	0.8	2.6	2.7	1.9	3.5
Strontium (SrO)	0.50	0.35	0.30	0.40	0.19	0.51	0.01
Barium (BaO)	0.35	0.35	0.30	0.30	0.34	0.45	0.23
Zinc (ZnO)	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Vanadium (V_2O_5)	0.02	0.02	0.03	0.02	0.02	0.01	0.02

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} (°C/h) = -0.6351xAsh – 0.1767 xNa_2O + 7.63

(R^2 = 0.98)

where, both Ash and Na_2O 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 R70

Sample	Actual R_{70} (°C/h)	Predicted R_{70} (°C/h)	Residual (°C/h)
3A	3.09	3.02	0.069
3B	3.01	2.96	0.042
4	3.39	3.46	0.079
5A	4.32	4.23	0.089
5B	3.66	3.75	0.095
6A	3.04	3.06	0.025
6B	2.94	2.94	0.003

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 R_{70} 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 (Na_2O) in one of the samples which did not conform to the observed simple linear R_{70} 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 R_{70}, ash content and sodium (Na_2O) content produced a self-heating rate prediction equation with an R^2 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.

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A STUDY OF THE FORMATION OF HYDROGEN PRODUCED DURING THE OXIDATION OF BULK COAL UNDER LABORATORY CONDITIONS

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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

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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.

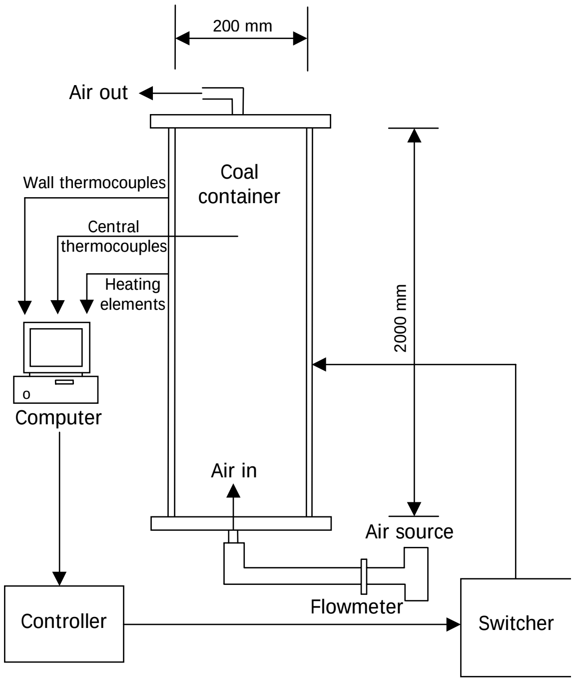

Figure 1 - Schematic of UQ two-metre column

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 56kg 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.

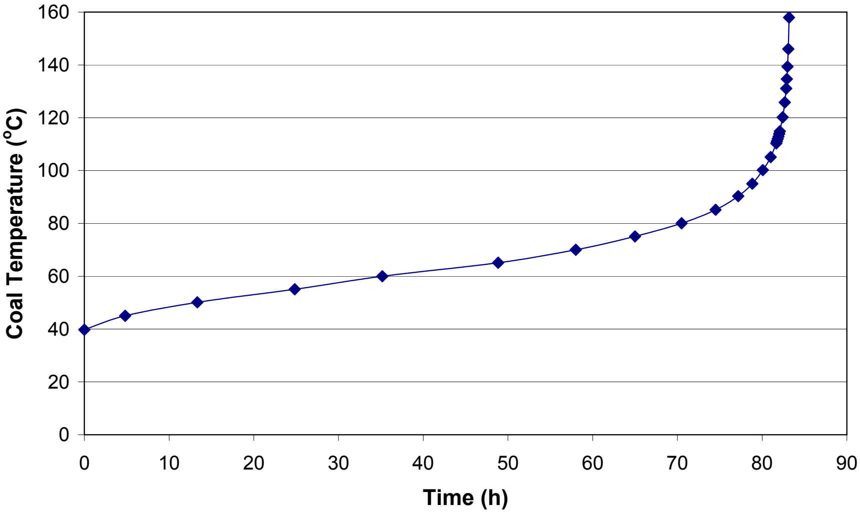

Figure 2 - Adiabatic self-heating curve for a Bowen Basin high volatile A bituminous coal

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 .

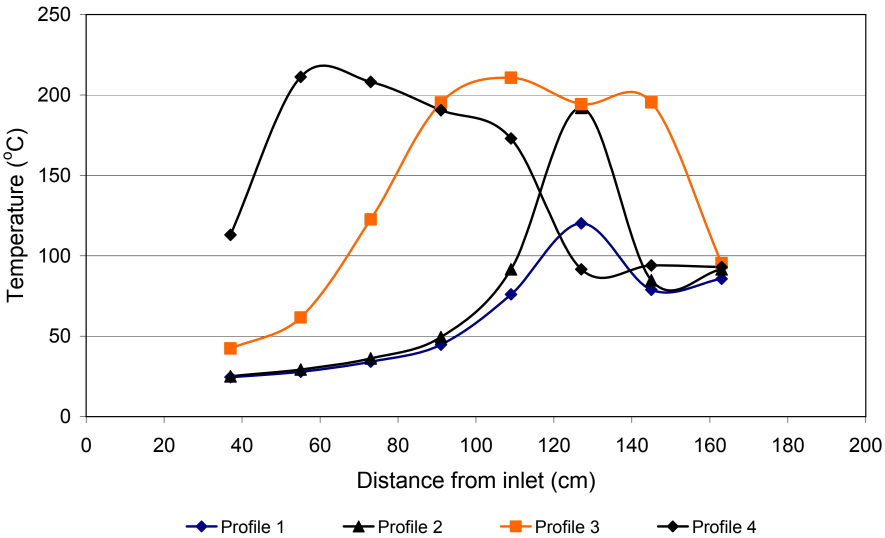

Figure 3 - Temperature profiles of the column for each gas sampling profile

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

	Exhaust	*Port 9*	*Port 10*	*Port 11*	*Port 12*	*Port 13*	*Port 14*	*Port 15*	*Port 16*
Profile 1	yes	-	-	yes	-	yes	-	yes	-
Profile 2	yes	-	-	yes	-	yes	-	yes	-
Profile 3	yes	yes	-	yes	yes	yes	yes	yes	-
Profile 4	yes	yes	-	yes	-	yes	-	yes	-

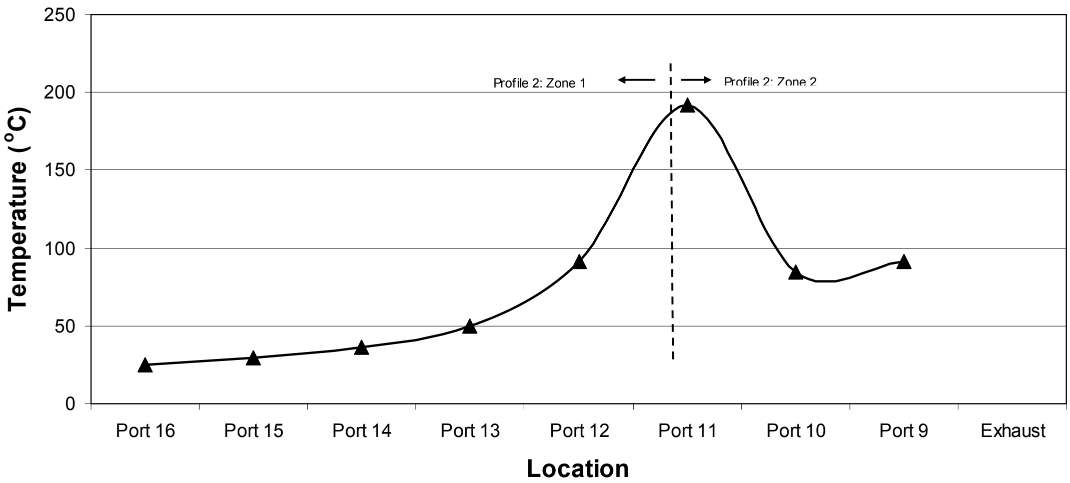

Figure 4 - Temperature profile 2 showing zones 1 and 2 based on hot spot location

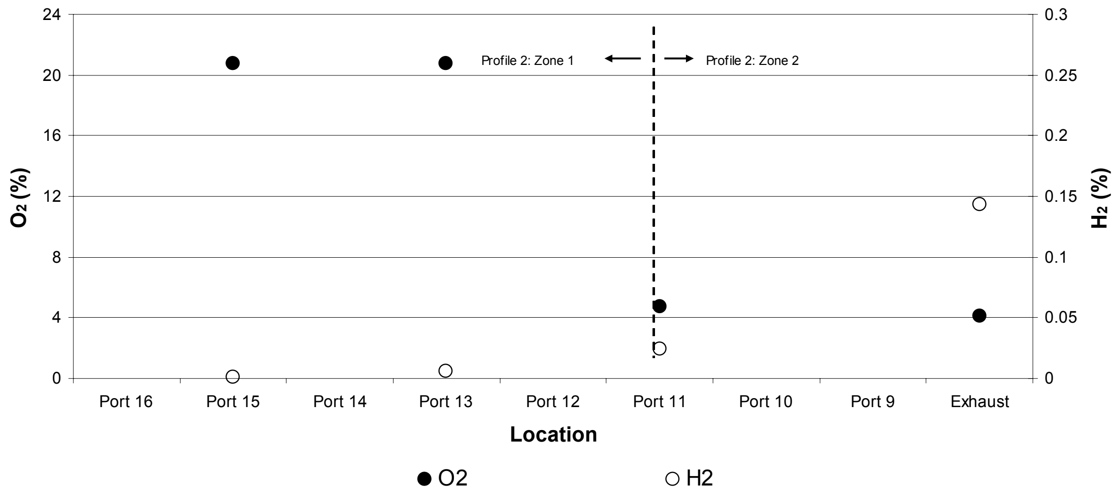

Figure 5 – Gas profile 2 showing oxygen and hydrogen concentrations

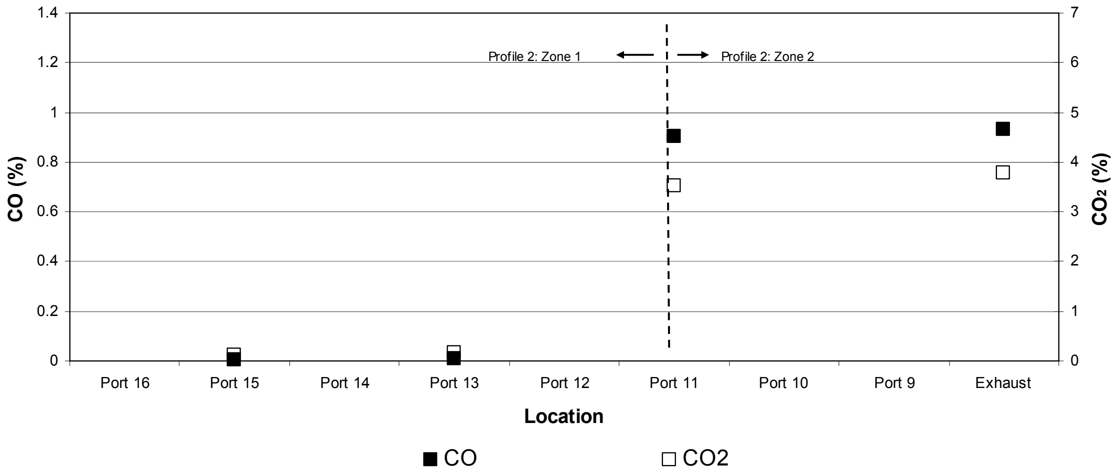

Figure 6 – Gas profile 2 showing CO and CO_2 concentrations

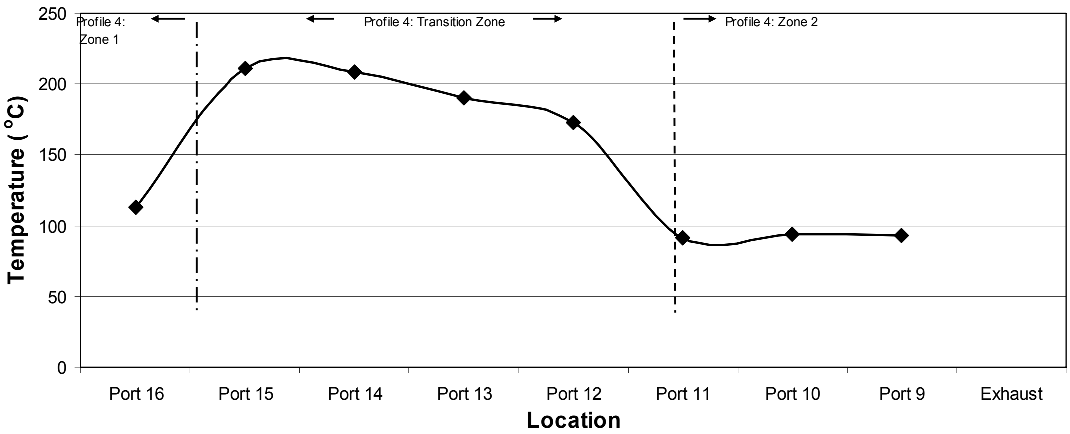

Figure 7 - Temperature profile 4 showing zones 1 and 2 based on hot spot location

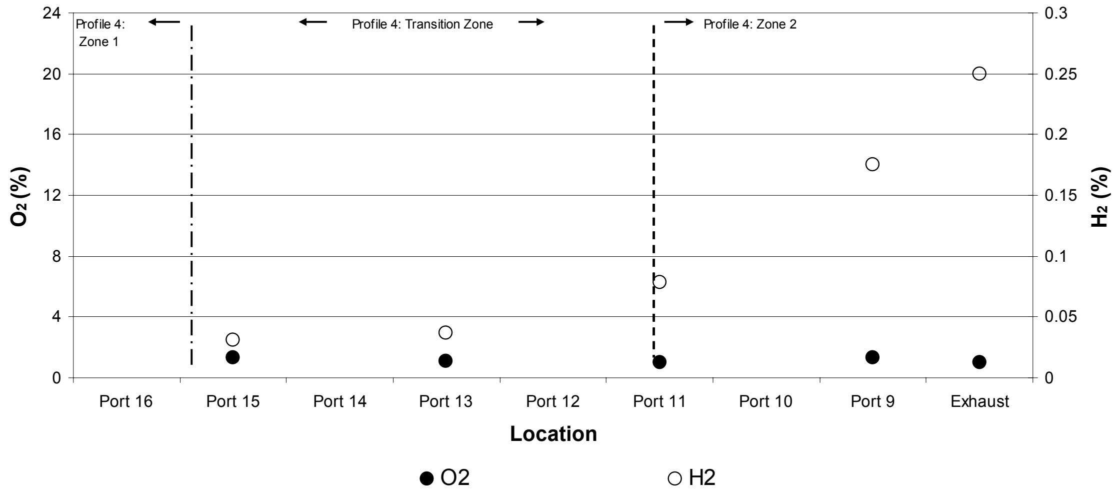

Figure 8 - Gas profile 4 showing oxygen and hydrogen concentrations

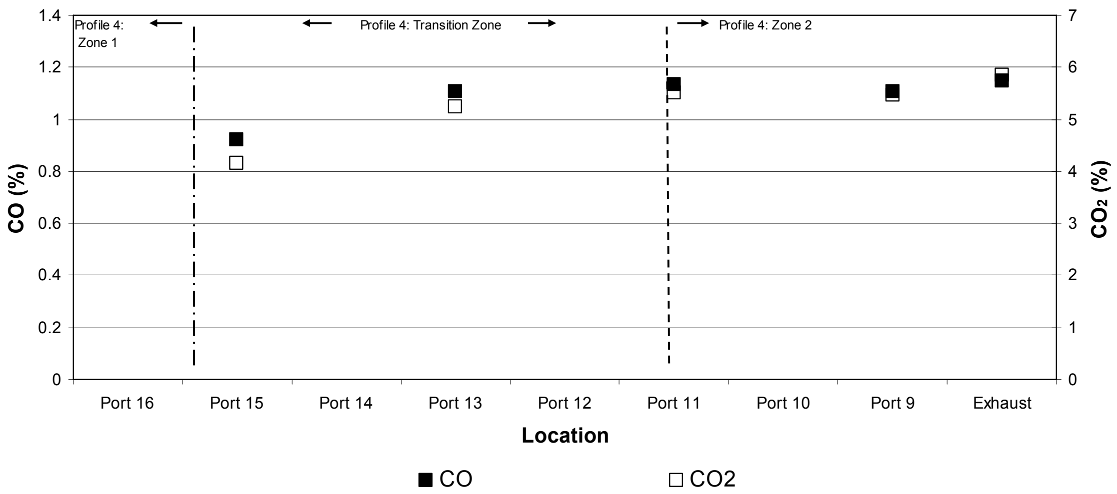

Figure 9 - Gas profile 4 showing CO and CO_2 concentrations

Figures 6 and 9 show the CO and CO_2 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.

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IDENTIFICATION OF SPONTANEOUS COMBUSTION PRONE ZONES IN LONGWALL TOP COAL CAVING GOAFS

Jun Xie[1, 2], Sheng Xue[2] and Weimin Cheng[1]

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^{o}. 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 m^3/s. A schematic of the panel layout is shown in Figure 1.

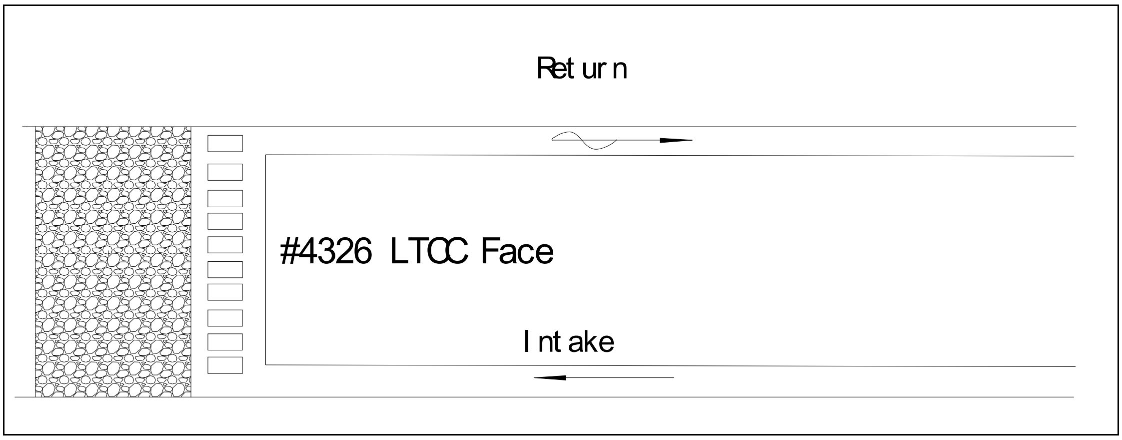

Figure 1 - Layout of #4326 LTCC face, Xinglongzhuang mine

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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).

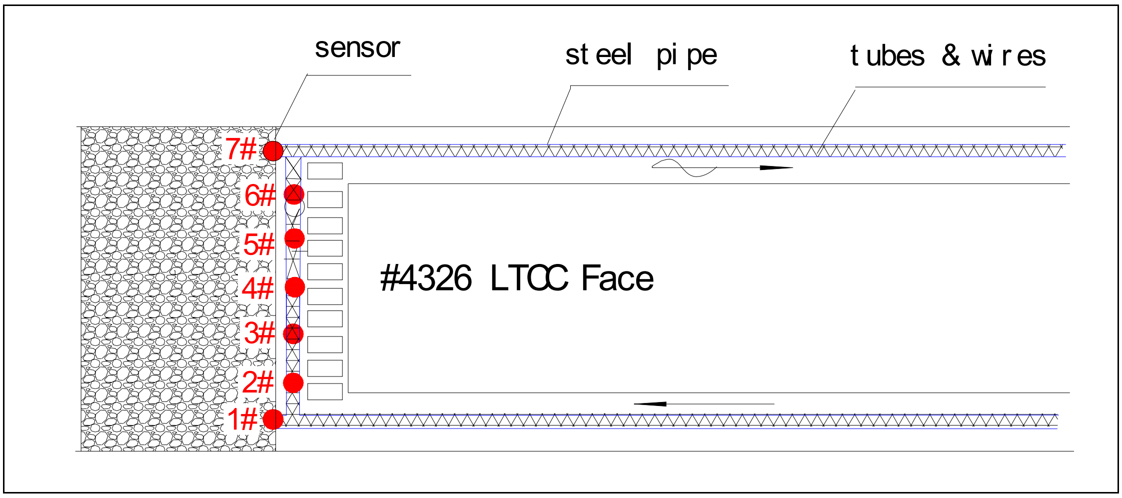

Figure 2 - Layout of measurement points at #4326 LTCC face

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.

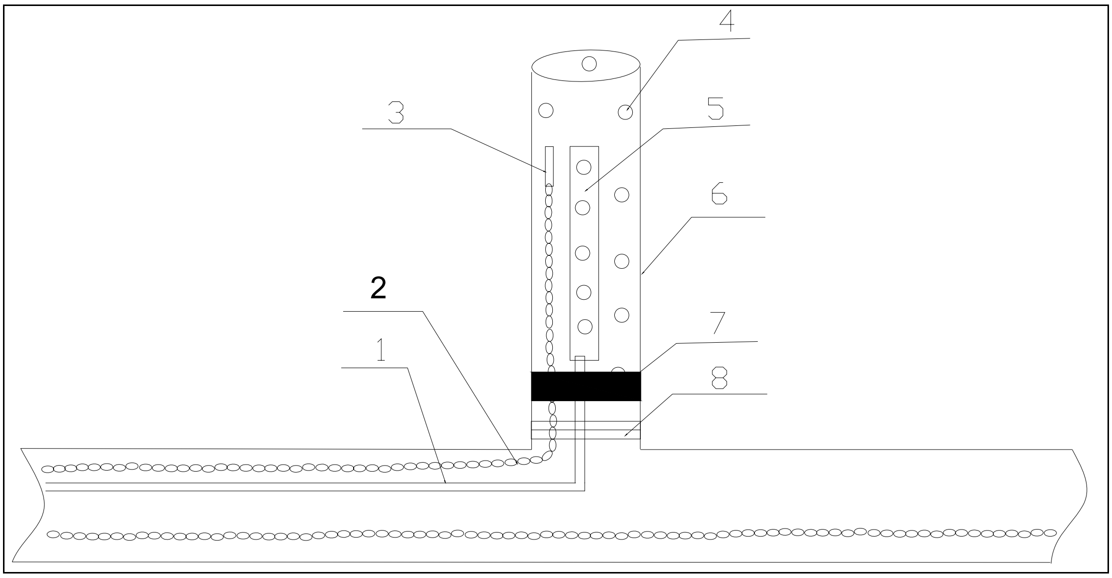

Figure 3 - Schematic of a measurement point

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 1 - Measured temperature and oxygen concentration in #4326 LTCC goaf

Distance inside goaf	#1		#2		#3		#4		#5		#6		#7	
	O_2	T	O_2	T	O_2	T	O_2	T	O_2	T	O_2	T	O_2	T
m	%	^{0}C	%	^{0}C	%	^{0}C	%	^{0}C	%	^{0}C	%	^{0}C	%	^{0}C
15	20.7	25.6	17.7	25.6	17.6	25.7	17.1	25.7	17.3	25.7	17.8	25.7	20.7	25.7
20	20.4	26.1	17.4	26.2	16.8	26.2	16.5	26.3	16.9	26.2	17.5	26.1	18.6	25.8
25	18.9	26.4	16.1	26.3	16.1	26.3	15.4	26.4	15.8	26.5	16.1	26.2	18.3	26.1
30	18.4	26.7	15.7	26.8	15.2	26.9	14.7	26.9	14.9	26.7	15.2	26.3	17.2	26.3
35	17.6	26.9	15.5	26.9	14.5	26.9	13.5	27.1	14.1	26.9	14.7	26.4	15.5	26.6
40	16.7	27.2	14.9	27.3	13.9	27.3	12.9	27.3	13.2	27.1	13.8	27.2	14.2	27.3
45	16.4	27.3	13.6	27.4	13.6	27.4	11.9	27.4	12.3	27.4	12.3	27.3	14.7	27.4
50	15.8	27.6	12.1	27.5	11.9	27.8	11.5	27.8	11.8	27.6	11.8	27.5	13.8	27.5
55	15.8	27.7	11.7	27.6	11.1	27.9	10.7	27.6	10.5	27.7	10.9	27.6	11.6	27.7
75	14.6	27.9	9.82	28.1	9.32	27.9	9.43	27.9	9.84	27.8	9.95	27.9	10.3	27.8
100	14.3	28.3	7.23	28.3	6.93	28.4	6.23	28.5	6.73	28.5	6.86	28.3	9.7	28.3
150	10.1	29.6	6.89	29.4	5.92	29.0	5.89	29.2	6.26	29.2	6.5	28.6	6.6	28.3
200	6.2	29.1	6.15	29.2	5.95	29.3	5.74	28.7	5.37	29.1	5.73	28.3	5.97	28.1

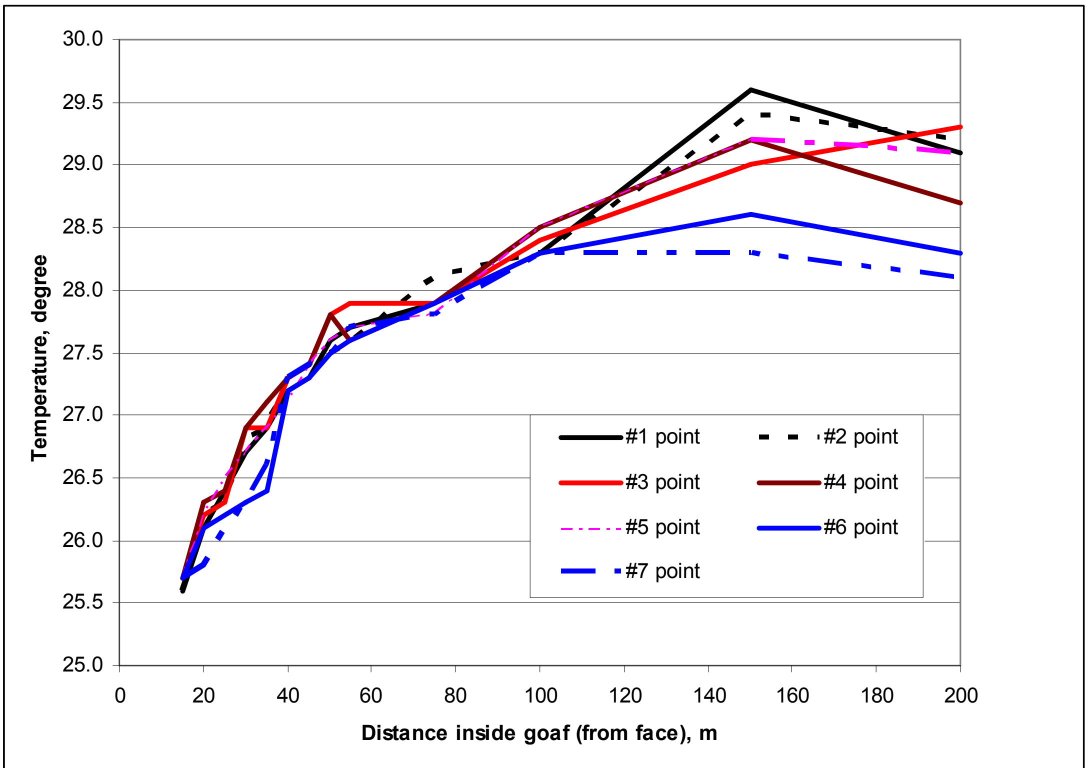

Figure 4 -Temperature variations inside #4326 LTCC goaf with face retreat

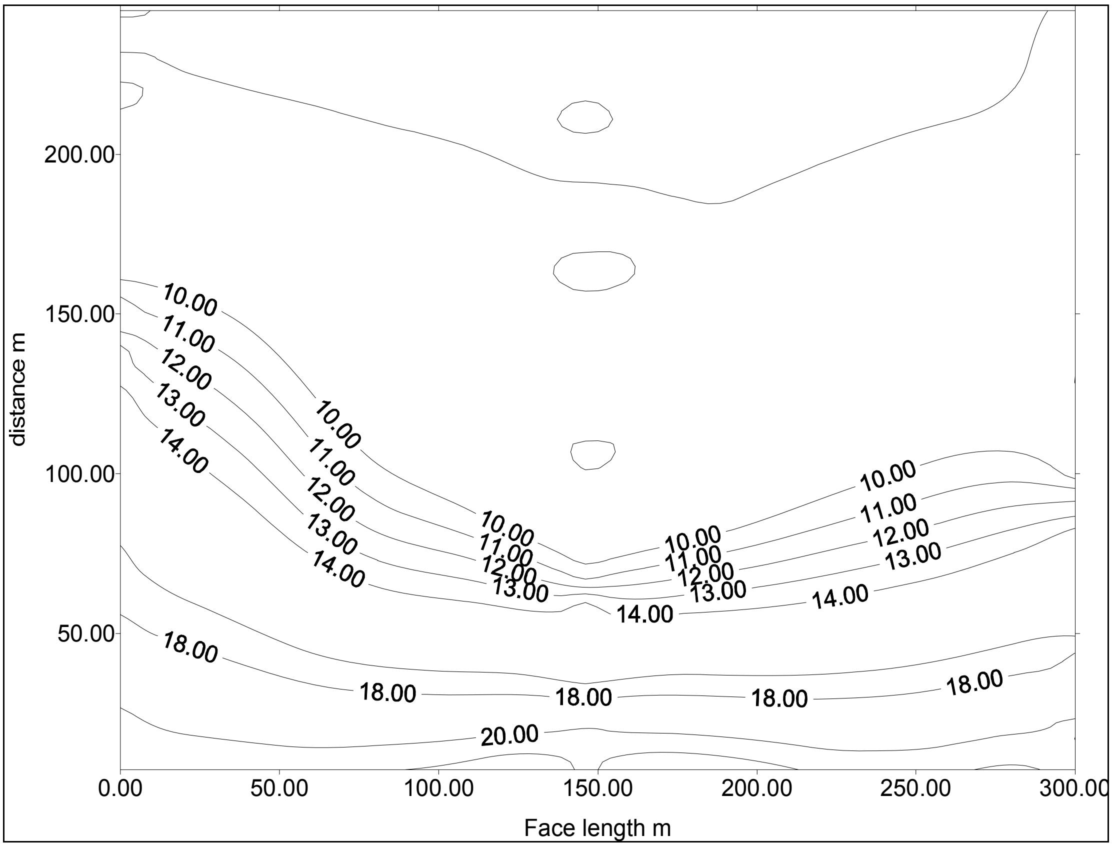

Figure 5 - Contour of oxygen concentration inside #4326 LTCC goaf

(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

Zone	high velocity	critical velocity	low velocity	width
goaf along intake gateroad	0-35.3 m	35.3-160 m	>160 m	10-20 m
goaf behind face	0-14.6 m	14.6-75 m	>75 m	260-280 m
goaf along return gateroad	0-28 m	28-100 m	>100 m	10-16 m

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.

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GAS DRAINAGE PRACTICES AND CHALLENGES IN COAL MINES OF CHINA

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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^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^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 1 - Drainage volume, ratio, and utilization ratio of top 35 gas drainage mines in China in 2002

No	Mine	Company	Drainage volume Mm^3	Drainage ratio %	Utilizing ratio %
1	Laohutai	Fushun	127.60	81.7	93
2	No.5	Yangquan	90.29	79.9	10
3	Houcun	Qinshui	47.86	54.6	-
4	No.2	Yangquan	33.92	29.5	90
5	No.1	Yangquan	33.82	47.9	64
6	Baijigou	Ningxia	30.80	53.0	100
7	Datong No.1	Songzao	27.66	43.5	95
8	Baijiao	Furong	26.58	26.5	100
9	Panji No.1	Huainan	25.16	38.0	-
10	Xinjing	Yangquan	23.73	48.8	100
11	Xieqiao No.1	Huainan	22.87	31.0	50
12	Luling	Huaibei	22.11	43.9	48
13	Tucheng	Panjiang	20.28	55.0	-

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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	Zhongliangshan	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	Moxinpo	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

Category	**Decay coefficient of gas flow from a inseam borehole of 100m in length, d^{-1}**	**Seam permeability* $m^2MPa^{-2}d^{-1}$**
Easily drainable	< 0.005	>10
Drainable	0.005~0.05	10~0.1
Hardly drainable	> 0.05	<0.1

* 1 $m^2MPa^{-2}d^{-1}$ 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 technque 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 m^3/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

Technique of gas drainage				**Applicable conditions**
Working seam drainage	Drainage without de-stressing	Pre-drainage of seam roadway development	Cross-measure boreholes drilled from rock roadway Inseam boreholes ahead of inseam roadway development	Outburst prone seam Gassy seam
		Pre-drainage of working face	Inseam boreholes	Outburst prone seam Gassy seam
			Cross-measure seam boreholes drilled from cross measure roadway, rock roadway or roadway in adjacent seams	Drainable seam Outburst prone seam
			Surface boreholes	Gassy and easily drainable seam Relatively shallow seam
	Drainage with de-stressing	Drainage while developing seam roadway	Boreholes ahead of development headings	Outburst prone seam Gassy seam
		Drainage while face retreating /advancing	Boreholes ahead of mining face	Outburst prone seam Gassy seam
			Cross-measure or inseam boreholes drilled from rock roadway ahead of mining face	Outburst prone seam Gassy seam
		Inter-crossing pre-drainage (measures taken to increase seam permeability)	Inseam boreholes Boreholes drilled from rock roadway or surface	Gassy and hardly drainable seam
Adjacent seam drainage	Drainage with de-stressing	Drainage of overlying and underlying seams	Cross-measure boreholes to adjacent seam	High gas emission from adjacent seam.
			Tunnel in adjacent seam for gas drainage	High gas emission from adjacent seam and normal boreholes can not cope with gas emission
			Surface boreholes	When surface borehole is considered to be a better option than underground borehole.
Goaf drainage			Pipes placed in goaf	No sponcom risk seam Sponcom risk seam where measures taken to mitigate sponcom risk
			Boreholes into goaf	
			Surface boreholes	

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 10^0-18^0, away from the return at an angle of 15^0-20^0, 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.

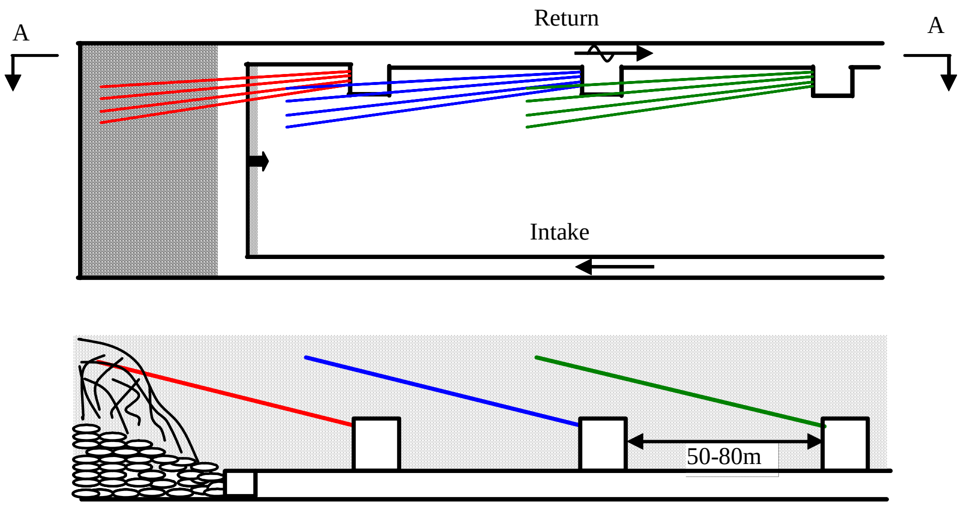

Figure 1 - Goaf gas drainage with cross-measure boreholes into fractured roof strata

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.

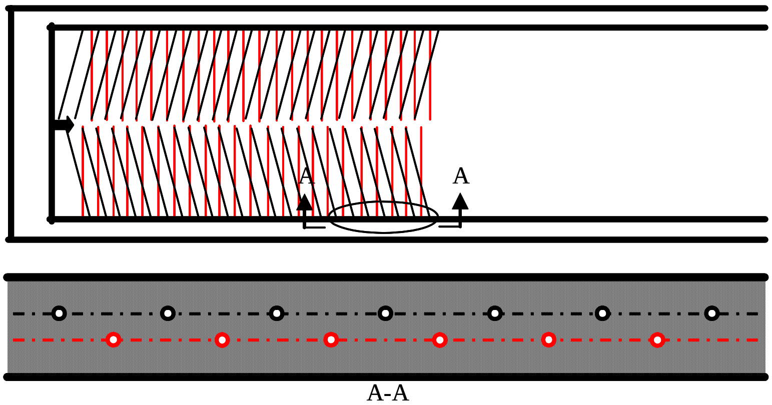

Figure 2 - Inter-crossing boreholes for inseam gas pre-drainage

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

Technique	Site	Seam parameters					Borehole parameters		Gas flow from 100m long borehole ($q=q_0e^{-Bt}$)		
		Thickness, m	Dip, °	Gas content, m^3t^{-1}	Gas pressure, MPa	Permeability, $m^2MPa^{-2}d^{-1}$	Diameter, mm	Spacing, m	Initial flow rate, q_0 ($m^3min^{-1}hm^{-1}$)	Decay coefficient B, $^{-1}$	Drainage Quantity, 10^4 m^3
Parallel boreholes	No.912 face of Yangquan No.1 mine	2.2	<10	18.3	1.3	0.015	73	1.7	0.005	0.004	0.204
Large diameter parallel boreholes							300	3.2	0.027	0.008	0.495
Parallel boreholes	No.42081 and 41041 faces of Jiaoxi mine	4.5 - 5.3	<10	17.7 - 14.9	1.2 - 0.8	0.55 - 3.6	75	4.8	0.084	0.009	1.426
Large diameter parallel boreholes							150	8.1	0.198	0.008	3.697
Parallel boreholes	No.13501 face of Jiaozuo Jiulishan mine	5.6	<10	16.9			65	2.4 - 3.0	0.040	0.008	0.711
Intercrossing boreholes								3.5 - 4.5	0.064	0.006	1.463
Parallel boreholes	$E_{9\text{-}10}$-20100 face of Pingdingshan No.10 mine	4.2	<10	13.5		0.56	75	2 - 4	0.028	0.029	0.139
Intercrossing boreholes								2 - 4	0.035	0.025	0.202

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 retreating 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