Proceedings of the 2006 Coal Operators' Conference

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FOREWORD

It is a pleasure to host the Coal Operators Conference in Wollongong once again, after its successful 2005 run in Brisbane, which was organised by the Southern Queensland Branch of AusIMM.

This year more companies and organisations are supporting 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 and researchers in the diverse field of coal mining technology. The conference is now entering its eighth year of existence and remains focused on issues of importance that are challenging to the industry with respect to coal mine safety, productivity and management adaptability.

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,
- Elena Di Stefano and Peter Vrahos and their colleagues at the UniCentre of the University of Wollongong for the management and registration of the conference. Peter is to be congratulated for setting up the Coal 2006 Conference web site,
- Leonie McIntyre of the Faculty of Engineering, University of Wollongong for typesetting the conference proceedings,
- Barry Robertson for audio–visual management of the conference venue, and
- The staff of Wollongong University Printery for printing the conference proceedings and to Gerard Toomey for designing the proceedings covers.

Finally, we wish to acknowledge the generosity of the sponsors and exhibitors for their financial contributions which enabled this conference to be held at affordable rates.

Naj Aziz (Conference Convener and Editor)
Walter Keilich, (Co-editor)
IN REFACE

In recent times the word “sustainability” has been subject to a range of creative and politically correct manipulation and adaptation. In numerous instances it has been redefined, modified and manipulated to suit a range of stories, scenarios and arguments, all tailored to imply that any particular development and or activity is fundamentally important to the continued existence of civilisation as we know it, or to the future of the world and the universe. Sustain according to the Shorter Oxford English Dictionary means “to support”.

Coal mining and the Illawarra Region is therefore a classic example of sustainability and the interdependence between a community and an industry. In 1847 when the Mount Keira coal mine commenced production, a fundamental bond between coal mining and the Wollongong community was initiated. Local communities, residential, commercial and local industries were established and flourished along the coastal plain from Helensburgh down to Wollongong and out to Dapto and these were all supported by coal mining. In this respect it is indeed fitting and appropriate that Wollongong is the venue for “Coal 2006 Sustainable Coal Development”.

As coal mining operations have progressed west of the escarpment into deeper, gassier sections of the coalfield, mining methods, techniques and technologies have evolved to meet the challenges the geological and mining conditions have dictated. It is therefore fitting that Coal 2006 provides an ideal opportunity for professional people with an interest in coal mining, to gather and consider new and imaginative ways, aimed at ensuring the coal mining industry in the Illawarra has a future and remains sustainable. The diversity of papers presented in Coal 2006 exemplifies the wide range of interests, areas of study and complexity of issues that confront modern mining operations and the ever changing community demands and legislative requirements.

Special mention and thanks go the authors and presenters of the papers, to the organising committees and to our sponsors especially the major sponsors BHP Billiton and Gujarat NRE, for without their generosity and support the success of the conference would not be guaranteed. As Chairman for the Illawarra Branch for the Australasian Institute of Mining and Metallurgy I welcome all delegates to “Coal 2006” and trust that the their deliberations will help support the future sustainability of coal mining, especially in the Illawarra.

Dr Chris Harvey
Chairman, Illawarra Branch
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GROUND CONTROL IN COAL MINES IN GREAT BRITAIN

James Arthur

ABSTRACT: In deep coal mining support of the underground roadway is fundamental and without knowledge of the mechanics of ground control a mine is unlikely to be operating safely and efficiently. It is therefore important for the support system to be designed satisfactorily for the size of the excavation required and to do so an understanding of the behaviour of the surrounding strata is required. Over the last 15 years there has been a major change in the mining industry and the support systems adopted. Roofbolting has been adopted as the primary support system, the industry has been privatised, legislation has been updated, a comprehensive research programme has been carried out and guidance documents produced.

INTRODUCTION

For many years the main roadway support system in Great Britain was steel frame or arch type supports. However, since the introduction of rockbolting systems in the early 1990s these have become the main support system for roadways and accidents in roadway drivages have been reduced. This reduction was attributed initially to the introduction of a code of practice, which introduced systematic monitoring and then later to the introduction of a simple and easy to erect mesh cage system at the face of roadway drivages. The system has proven to be so successful that it has been introduced into roadways supported by square set steel type supports. A working group is currently reviewing support in large roadways supported by steel arch supports to design a mesh cage system for this type of roadway.

In 1993 a major accident at a coal mine resulted in the deaths of three people. The subsequent hearing recommended that ‘A national research programme in rock bolting technology, the associated geology, improved instrumentation for monitoring, effects of pillar edges, gate side packs, pillar sizes etc. should be agreed between the industry and the universities with a rock mechanics capability’ (Crossland, 1994). The Health and Safety Executive (HSE) sponsored a comprehensive research programme to address this recommendation. This research has led to improvements in monitoring techniques, rockbolt materials, consumables, and a better understanding of the strata conditions and support mechanisms.

MINING CONDITIONS

Mining takes place at depths of up to 1100 m, the typical depth being 800 m to 900 m. Many of the mines are over 100 years old. Many seams have been worked and therefore interaction between seams can be a major problem. The geology of the rock above the coal seams is variable, ranging from weak mudstones up to 5 m in thickness grading up to siltstone and sandstone with a compressive strength, which can exceed 60 MPa. The stress field is a major factor in dictating mining conditions. The magnitude and direction of the major horizontal stress component can have a major influence on roadway failure, (Altounyan and Hurt, 1998)

HISTORY OF ROCK BOLTING

Investigations into the use of rockbolting as a support system were being carried out by Her Majesty’s (HM) Inspectors of Mines as early as 1954. They researched the use of roof bolts in mines in the USA and they emphasised the need for care and experiment where bolts are to be used and concluded ‘that roof bolts should not be used as the only means of support in roadways likely to be affected by future workings, where natural weaknesses in the strata are present and where bed separation occurs’, (Hodkin and Lawrence, 1954)

Rockbolting was introduced in 1964 using mechanical point anchored bolts but following a number of roof falls the use of these bolts ceased. Further trials using resin anchored bolts took place in the 1970s and early 1980s. They were found to be unsuitable for the conditions due to their low strength and a lack of understanding of the forces involved. In the mid 1980s a review of current bolting techniques throughout the world took place. The systems that were in use in the United States and Australia were trialled. The Australian bolting system, utilising high strength rockbolts fully bonded with polyester resin were found to be the most effective for British mining conditions.

1 HM Mines Inspector of Mines, UK
Due to the previous failures with rockbolts and because the majority of the mines are deep and use single entry roadways (up to 3000 m in length) for longwalls it was felt that the system must be introduced in a controlled manner via an extensive programme of research and development. The development of a code of practice was also seen as essential to enable the rockbolted system to be introduced safely. It was also essential to build and maintain the confidence of the miners, supervisors and Trade Unions and to provide comprehensive training to ensure that the installation standards were high. Rockbolting was safely and successfully introduced into coal mines as a primary support system in the 1990s. Today approximately 95 per cent of all new roadways are supported by rockbolt systems.

**PRIVATISATION OF THE COAL INDUSTRY**

In 1994 the government announced that the nationalised coal industry (British Coal Corporation) was to be privatised and at the end of 1994 the coal mines were transferred to the private sector.

Before taking the decision the government consulted with the Health and Safety Commission and sought advice on how the industry could be effectively regulated. The government stated its commitment to safety, recognising that the maintenance of safety standards was of prime importance. The Health and Safety Commission provided a report to ministers in October 1993 and in this report recognised that ‘the legislation be flexible enough not to restrict the introduction of new technologies or new methods of work, but still ensure that health and safety was not prejudiced’ (HM Government 1993)

The Commission recommended the formation of the Deep Mine Coal Advisory Committee (IAC) and that key guidance be progressively adapted and updated. The Chief Inspector of Mines chairs this committee, which has subsequently been renamed The Mines Industry Advisory Committee (MIC). Representation includes employers, employees, mines rescue, healthcare providers, equipment suppliers and contractors.

The updating and replacement of British Coal guidance with official guidance is managed through working groups made up largely of MIC nominees. One of these working groups is the ‘Support Guidance Working Group’.

**LEGISLATIVE RENEWAL**

The Mines and Quarries Act 1954 contained the requirement to control strata and secure working places. The Coal and Other Mines (Support) Regulations 1966 that applied to coal and stratified mines specified minimum and maximum distances for a range of support systems. These regulations were detailed and complex and with the introduction of newer technologies many exemptions were required. This was particularly the case when rockbolting was introduced.

A fundamental review of legislation was carried out in 1970 and the review was critical of prescriptive law, which was difficult to change. The review was the foundation for the Health and Safety at Work etc Act 1974 (HSWA) (HM gov, 1974). This act came into force on 1 January 1975 and provided for the introduction of new regulations that set broad objectives to be achieved, to progressively replace older detailed and prescriptive legislation. These regulations are supported by codes of practice and guidance documents.

Under the HSWA 1974 the Coal and Other Mines (Support) Regulations 1966 were updated and the Control of Ground Movement Regulations (CGMR) 1999 introduced, Figure 1. The regulations differ from the 1966 support regulations in that they apply to all mines, not just coal and stratified mines.

The CGMR 1999 places a general duty on the manager to ensure the safety of the mine, assess ground conditions, design the ground control measures, draw up ground control rules, notify new ground control proposals in coal mines to the Mines Inspectorate of HSE, ensure that the rules are implemented and assess the effectiveness of the ground control measures.

There is a duty on the mine manager to ensure that before any excavation is undertaken an assessment of ground conditions must be undertaken. The assessment should take into account the geology, rock properties, stresses, extent of ground to be controlled, possible failure mechanisms, and effects on other working places, environment and historical data.
There is a duty on the manager to ensure that a design document has been prepared, which, takes into account the assessment of ground conditions and describes the ground control measures which are to be undertaken to keep places in the mine secure. The design should include the limits of extraction, excavation dimensions, pillar sizes, support density, support materials, methods of work, abnormality procedures other known risks such as faults.

There is a duty on the manager to ensure that the design document is turned into practical instruction, direction and guidance. From this information support rules are constructed that show the ground control measures described in the design document and instructions on how to implement them safely. Contained within the rules are details of support materials and equipment, methods of work, support density, sequence of excavation etc.

There is a duty on the manager to ensure that an appropriate monitoring system/scheme is introduced that ensures that the adequacy of ground control measures is being assessed at all times. The scheme must be appropriate for the circumstances and should allow time for action to be taken to recover and stabilise the situation. The results must be recorded and the system regularly reviewed. The systems adopted for roof bolt support systems are by tell tale, Figure 2 and extensometers, Figure 3.

There is a duty on the manager to notify the HSE, in writing no less than 28 days prior to making the change, if he wants to make a significant change to any ground control measure.
IAC GUIDANCE/STANDARDS DOCUMENTS

The Support Guidance working Group developed the first code of practice “Guidance on the use of rockbolts to support roadways in coal mine”, Figure 4 (Deep Coal Mines Advisory Committee, 1996a). It provides guidance on the safe use of rockbolts to support roadways in coal mines. Other guidance documents have been developed for cable bolts (Deep Coal Mines Industry Advisory Committee, 1996b), flexible bolts, (Deep Coal Mines Industry Advisory Committee, 2000), lifting bolts and passive supports in coal mines (Deep Coal Mines Industry Advisory Committee, 2002).

The rockbolt systems used utilise consumables that comply with a prescribed performance standard. The consumable items specified in the code of practice must meet criteria specified in the British Standard, BSI 7861 Pt 1 and 2, (British standards 1996a and 1996b). This standard applies to both rockbolt and cable bolt/long tendon systems. Due to improvements in technology and consumables BSI 7861 Part 1 has been updated, and was currently going through the consultation process. BSI 7861 Part 2 was in the process of being updated.

Guidance on the use of rockbolts to support roadways in coal mines

The guidance document (Deep Coal Mines Industry Advisory Committee, 1996a) covers the following matters:

1. definitions,
2. classification of the geotechnical assessment and site investigation,
3. support system design,
4. design verification monitoring,
5. routine monitoring and recording scheme,
6. training, and
7. list of consumables.

Contained within the annexes are tests for bond strength; short encapsulation pull tests; descriptions of telltales and extensometers; training required for workmen, officials, roof bolting staff and managers.

The guidance document is not suitable for places in coal mines such as goaf scours; gate roads serving advancing faces; cross measure drifts and headings formed by shotfiring off the solid.

A supplementary guidance document on the use of flexible bolts in reinforcement systems for coal mines has also been constructed (Deep Coal Mines Industry Advisory committee, 2000). This document is to be read in conjunction with the above rockbolt guidance document.
Geotechnical Assessment and Site Investigation

To carry out the assessment the manager, if not suitably qualified or competent should appoint a design engineer, who should be a chartered engineer or equivalent, who has had three years experience in work related to mine strata control.

The assessment should define the area covered and take into account all factors which are likely to affect the performance of the support system throughout the life of the roadway.

The site investigation should include reference to the following:

1. geology
2. stress,
3. pillar design and effects, etc,
4. environmental effects,
5. bond strength and
6. stand up time.

Support System Design

Where the assessment indicates that the stratum is suitable for the use of rockbolts a support design needs to be prepared. The design engineer will prepare the design on the basis of the results of the site investigation. As a minimum the design should take account of the following:

1. profile of the heading,
2. length of the rockbolt - minimum 1.8 m,
3. density of rockbolts - minimum 1 bolt/m²,
4. placing of rockbolts,
5. type of rockbolt,
6. holes drilled for rockbolts (finished hole diameter not to exceed bolt diameter by more than 7 mm) and
7. the system of support for the roadway sides.

Design Verification Monitoring

The initial design of the support system needs to be verified by comprehensive monitoring, which includes detailed measurement of roof dilation and rockbolt loads. A station is normally set up with at least four x seven meter length strain gauge bolts or sonic extensometers installed across the roadway that can measure a dilation of at least 1 mm over 15 points on the bolt. This installation is set up at the entrance to the intended development. If the drivage is intended to be of a long length then ideally another station should be set up inbye, normally prior to turning onto the face line. However, the guidance document does not require more than one station to be installed.

Routine Monitoring and Recording Scheme

The manager requires a scheme for the routine monitoring of roadways and should appoint a suitable qualified person to implement, audit and co-ordinate the scheme. This person would be called the rockbolting co-ordinator and be qualified to Higher National certificate standard in a mining related subject.

The scheme should describe the routine monitoring devices, the minimum is dual height tell tales every 20 m and extensometers every 200 m. The manager’s scheme must set action levels and for rockbolted roads in coal mines the maximum roof dilation allowed is 25 mm before remedial action must be taken. In some mines the action levels are set lower than 25 mm due to localised conditions, lack of horizontal stress.

A plan of all roadways supported by rockbolts should be prepared and a schedule of measurement zones and frequency should also be prepared.

All workers involved with rockbolting need to be suitably trained. The appendix to the document should describe the minimum training required for workmen, officials, rockbolt co-ordinators and managers.

Guidance on the use of cable bolts to support coal mines.

In coal mines cable bolts are used as a secondary support system to improve conditions when roof bolts systems are beginning to fail. A guidance document has been constructed for the use of cable bolts (Deep Coal Mines
Industry Advisory Committee, 1996 b). The guidance applies to situations where cable bolts are installed as additional support when excessive strata movement is experienced in places principally supported by rockbolts. The document is constructed along the same lines as the guidance document for rockbolts, i.e. assessment, design, monitoring and training. Tell tales are used as monitoring but they must be installed to at least the height of the cable bolt length +1m.

Where cementitious grouts are used it is important that the liquids to solid ratio of the mixed grout is accurately measured to ensure the correct consistency for both pumping and strength.

Skilled workmen are used to install cable bolts.

**BSI 7861 Strata reinforcement support system components used in coal mines – Part 1 Specification for rockbolting**

This part of BS 7861 specifies dimensional, material and performance requirements for rockbolting support system components use in coal mines (British Standards, 1996a). Included are steel rockbolts, glass reinforced plastics (GRP) rockbolts, resins, nuts, conical seats and domed washer plates.

The standard defines for example, minimum bond strength for a rockbolt/resin/rock system and system stiffness, nut breakout facility, equivalent diameter of the bar and gel setting times.

The standard describes the composition of the rockbolt. This should be made from steel with a homogeneous structure having a chemical composition with maximum of carbon 0.3 per cent, manganese 1.6 percent, sulphur 0.05 percent and phosphorous 0.05 percent.

The rockbolt has a minimum yield strength of 640 N/mm² with a minimum elongation after fracture of 18 per cent.

One of the most important changes to the standard was a fracture toughness requirement. Research, described later, has shown that an average charpy value of 27 joules is required to give the bar the desired fracture toughness properties.

**RESEARCH AND DEVELOPMENT**

A Health and safety Executive sponsored programme has covered research into the following:

1. Rockbolt metallurgy,
2. Mining systems,
3. Roadway stability,
4. Risk assessment techniques, and
5. Instrumentation.

The main falls of ground and related major injuries in rockbolted roadways have been due to, broken or corroded rockbolts, collapse of the ribside, failure to monitor correctly/ respond to monitoring or failure to install remedial support.

**Rockbolt Metallurgy**

In the early 1990s a number of falls occurred in roadways supported primarily by rockbolts. It was found that a number of the rockbolts had broken. These broken bolts were examined at the Health and Safety Laboratory (HSL). It was found that most of the failures were associated with bends in the bolts but all of the broken bolts had corroded to some degree. The fractures had initiated at the root of a corrosion pit and the fracture surfaces indicated brittle failure. Some fracture surfaces were heavily corroded indicating that failure had occurred some considerable time before roof collapse. Indications were that the original rockbolt materials were sensitive to the presence of defects. With corrosion pit defects the material changed from plastic failure with good elongation to sudden brittle failure without any elongation.

Corrosion studies were carried out and the results indicated that a small amount of corrosion could have a significant effect on a bolt. The tests showed that the corrosion depth must be at least 6.25 times the radius at the base of the pit to cause brittle failure with little or no plastic deformation. The depth of corrosion pit that was needed to cause brittle failure in the original type bolts was found to be 1 mm.
It became clear that the rockbolts possessed a low toughness. A modified steel was produced that increased the fracture toughness of the bolt threefold whilst still complying with the chemical and tensile strength requirements of BS 7861 Part 1. Following improvements to the fracture toughness of the steel, the critical defect size of the corrosion pit has been increased to 3 mm. It was thought that this is unlikely to occur in normal conditions. It was also found that the presence of hydrogen sulphide increases the tendency for brittle corrosion cracking occurring.

Fracture toughness tests are expensive to carry out therefore research has been carried out to determine a correlation between Charpy impact values and fracture toughness. A minimum Charpy impact energy requirement of 27 joules was recommended. This minimum value has been incorporated into the updated part 1 of the BSI 7861 document. The Charpy value is a surrogate, cheap and simple, indicator of fracture toughness, which is a much more scientific but also more difficult and expensive to carry out.

**Risk assessment techniques**

Experience with rockbolting under high stress conditions has shown that, even with good design procedures incorporating numerical modelling and detailed monitoring, it remains difficult to predict, during drivage, the behaviour of gate roadways under the influence of increased stress generated by the retreating longwall face. The use of risk assessment techniques was used to categorise areas with an increased risk of instability and to comply with modern health and safety legislation, which requires that an assessment of risk be, carried out in mine roadways. The initial work was carried out at Thoresby Colliery (Cartwright and Bowler, 1999) leading to a practical assessment technique being developed. The main principles are: identify the falls of ground hazards, identify the mechanisms that lead to the hazard, incorporate them into a survey sheet, examine the roadway in detail, and complete the survey sheet. Input the data into a spreadsheet, and a risk assessment is produced for the roadway identifying areas that require attention. This approach is routinely applied in coal mines.

**Laboratory short encapsulation pull test**

System performance has been measured by the ‘double embedment test’, that is defined in BS 7861 Part 1 1996. The test involves axial loading of a tendon that has been grouted into two undersized steel tubes; the inner walls of the tubes are threaded to give a good bond. When the steel tubes are pulled apart in a tensile testing machine, the load transfer capability of the resin/bolt (or cable/grout) interface is tested. The bond length used in the test is designed to ensure that bond failure occurs below the yield load of the steel. The problem with the double embedment test is that in practice bond failure can occur at the resin/rock boundary. Research has enabled a Laboratory Short Encapsulation Pull Test (LSEPT) to be developed. This test enables measurement of the system performance at the resin/rockbolt interface to be measured in the laboratory. It consists of confining a sandstone cylinder using a biaxial cell to simulate the stresses imposed underground. A hole is drilled in the sandstone cylinder and a rockbolt inserted through resin into the hole. The system is then pull tested and the bond strength measured. The LSEPT has been incorporated into the updated BSI 7861 Part 1 document.

**Lifting and suspending from rockbolts**

Equipment is lifted and suspended routinely from the roof in roadways supported by rockbolts. Research was carried out into the effects of lifting and suspending from these anchor rockbolts. Results showed that the most likely failure mode for anchor bolts used for lifting is bond failure due to poor installation, or loss of resin on installation. Anchor bolt failure due to bolt deformation associated with subsequent roof movement is also possible, particularly where the immediate roof shear results in significant bolt bending. The guidance on the use of lifting and suspension of equipment in rockbolted roadways was updated after this research.

**Ground control at small coal mines**

Small shallow mines have different support problems to those experienced in the larger and deeper mines. A guidance document was developed for small coal mines that would enable the small mine owner to benefit from the techniques developed for large mines.

**Instrumentation of flexible bolts**

Flexible bolts, typically 4 m length, are constructed of a number of cable strands, which are installed in polyester resin. They are installed in the same diameter hole as rockbolts (27-28 mm). This resulted in longer tendon reinforcement being installed at the face and consequently less reliance on cable bolting as a remedial tool. Flexible bolts are routinely used in combination with rockbolts to support the face of drivages as part of the primary support system. With the introduction of flexible bolts at the face new instrumentation techniques had to be developed to extend the basic monitoring principles previously applied only to rockbolts.
Triple height tell tales were introduced to allow monitoring of the area above the rockbolt to distinguish movement occurring within the rockbolted height and movement in the section supported by the flexible bolt. This is a simple device which extends the dual height tell tale concept to three heights, Figure 5. The A indicator shows movement below the top of the rockbolt, the B indicator shows movement between the top of the rockbolt and the top of the flexible bolt and the C indicator showed movement above the flexible bolted height. The triple height tell tale is used nearly as frequently as the dual height tell tale in coal mines.

Fig. 5 - Triple Height Tell Tale

Mesh cage support system

Where on board bolter miner systems cannot be used a mesh cage system of temporary support is used. The mesh cage system of support, Figure 6, has been adopted in most coal mines where rockbolts are used as the primary support system. It has proven to be so successful that it has been adopted in roadways that are passively supported with steel delta type supports.

Fig. 6 - Mesh Cage

The mesh cage system enables workmen to work under supported ground at all times when at the face of the heading. A mesh panel is pushed to the roof using two stinger air leg machines and another mesh panel is hung vertically from the roof at the face. The side mesh is unfolded from the roof mesh and draped down the rib side and fixed to the previous panel. The roof and rib bolts are then installed. When the bolting cycle is completed the remaining unbolted mesh panel is tied back horizontally to the roof ready for the next cutting cycle.
A working party is currently looking at ways that this system can be adapted for use in large roadways supported by steel arches.

**FALLS OF GROUND ACCIDENTS**

The introduction of rockbolts as a primary support and the introduction of the mesh cage support system have enabled accidents caused by falls of ground to be reduced. The last fatal accident that occurred in UK coal mines caused by a fall of ground was in a heading supported by passive arch supports in December 2001. If you restrict this to roads supported primarily by rockbolts then the last fatal accident was in September 1997 at Riccall mine, due to a ribside collapsing. If you further restrict that to fatalities caused by a fall of roof in rockbolted roads then there has not been a fatal accident since 1993.

In 1995/6 there were no fatal accidents and a total of 29 major injuries caused by falls of ground. Eleven were in roadways supported by rockbolts and 18 in roadways supported passively. The mesh cage was introduced as a temporary support system in roadways supported by rockbolts in 1996/7 and the major injuries reduced to three.

Table 1 (Health and Safety Executive, 2005) shows the accident rates/100,000 work shifts for fatal (F) and major injuries (MI) caused by falls of ground from the reporting year 1999/2000 to 2004/2005.

<table>
<thead>
<tr>
<th>Table 1 - Fall of ground accident rates per 100,000 man shifts</th>
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</thead>
<tbody>
<tr>
<td>Type of support</td>
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<tr>
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<tr>
<td>Rockbolt</td>
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<tr>
<td>Passive</td>
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Approximately 95 percent of the roadways in UK coal mines are supported by rockbolts yet less than 20 percent of the accidents occur in these roadways.

**CONCLUSIONS**

The updating of the legislation, introduction of guidance documents and improved temporary support systems along with the results from a comprehensive research programme have been fundamental in improving underground safety and efficiency. This has brought the industry much nearer to being able to choose the appropriate support system for any particular application.

**ACKNOWLEDGEMENTS**

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**REFERENCES**


Health and Safety Executive (HSE), 1999. The siting and support of coal mine roadways in the vicinity of old wastes, contract research report 3381/R72.018.


Health and Safety Executive, 2000. Stability and support of the sides of mine roadways, HSE internal research documents. Research Project ref 4086/R33.081.


Cartwright, P and Bowler J, 1999. The development and use of risk assessment techniques to assess roadway stability in the Parkgate seam at Thoresby Colliery, Proc. 18th Conf. On ground Control in Mining Morganstown, USA Aug, P 72-81

Health and Safety Executive, 2001. The effects on coal mine roadways of lifting and suspending from rockbolts, HSE contract research report 3686/R33.048


Reporting of Injuries, Disease and dangerous Occurrences Regulations 1995. HSE books ISBN 0 7176 2431 5

Health and Safety Executive, 2005, HSE statistics [on line]. available from: www.hse.gov.uk
Rapid Rating Using Coal Mine Roof Rating to Provide Rapid Mine Roof Characterisation from Exploration Drilling

Justine Calleja

ABSTRACT: The Australian coal industry is currently experiencing rapid expansion with many companies fast tracking the development of new mines. In many cases, operators are collecting large quantities of exploration drilling data for pre-feasibility and feasibility studies as well as for operational start-up and mine expansion.

“Rapid Rating” is a new method for calculating the Coal Mine Roof Rating (CMRR). It has been designed to allow large quantities of exploration data to be processed quickly to provide a standardised indicator of geotechnical conditions. It is intended to make CMRR more readily available to mine operators. When the CMRR was first developed there was less core drilling conducted in the USA than is conducted in Australia today. So it was specifically designed to allow geotechnical information to be collected easily in the absence of core data. It was then modified to allow calculation from core. The method for calculating CMRR required a geologist to collect the inputs manually, and this may take between 1 – 4 hrs. The need to assess large quantities of drill core in very short time frames has only emerged relatively recently, and “Rapid Rating” has been developed to meet this need. “Rapid Rating” can calculate a CMRR from between 40 minutes and 5 minutes for a large data set. It can calculate CMRR over numerous possible bolt lengths for sensitivity studies on different bolting horizons and during the early stages of mine design when bolt length is still a variable. The “Rapid Rating” calculation method is automated which makes the results more repeatable and less subjective. The “Rapid Rating” system, is described, which explores the benefits and limitations of this technique for mine design and strata management.

What is CMRR

The Coal Mine Roof Rating is an empirical method for quantifying the engineering properties of mine roof. The CMRR (Mark and Molinda, 1994) weighs some of the geotechnical factors which may affect the competence of mine roof and combines them into a single rating on a scale from 0 to 100.

Rock mass classification systems (such as Bieniawski’s “RMR” and Barton’s “Q”) have been used for many years in geotechnical engineering. These systems were not developed specifically for coal mine roofs and thus are less applicable to stratified layers (as a result of sedimentary geology). The bedding of coal mine roof is often an important parameter on the competence of the roof.

The RMR and Q systems tend to focus on the properties of joints rather than horizontal bedding. They also rate one unit at a time, whilst coal mine roofs typically consist of several layers. The CMRR has been developed specifically for coal mines based on a database from nearly 100 mines in every major coal field in the USA.

The CMRR makes it possible to compare ground control experience from different mines, coalfields, and countries. This makes it possible to collect a large database of case histories which can then be used to assist in the development of ground control strategies.

The CMRR can be calculated from roof rock exposures such as highwalls and overcasts or from exploration drill core. Colwell, 2003, describes the procedure to be used to calculate CMRR in Australia. The input parameters which are used to calculate CMRR from core are listed below with their approximate weighting (vary considerably depending on specific geology):

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1 SCT Operations Pty Ltd
• 70 % fracture spacing – the spacing of discontinuities such as bedding planes, joints and faults;
• 30 % UCS – uniaxial compressive strength of the intact rock;
• -20 % weak bedding planes – the number of weak bedding planes between units;
• -10 % moisture sensitivity – the propensity of the rock to degrade upon exposure to water;
• -20 % groundwater – the quantity of water flowing into workings;
• -5 % overlying weak rock – the presence of much weaker rock overlying the immediate roof.

While rock mass classification schemes such as the CMRR are appropriate for their original application, particularly within the limits of the case studies from which they were developed, considerable caution must be exercised when applying rock mass classifications to other rock engineering problems, or to cases which lie outside the database.

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. It does not include mining geometry such as roadway span or orientation of workings. It does not include roof support. It does not include rib conditions. As such it should not be used on its own for the purpose of geotechnical design (e.g. roof support, roadway spans, cut-out distances, pillar design etc). Empirical design systems such as ALTSII, use CMRR to determine design parameters but do not rely solely on CMRR. They also include additional input factors such as stress and geometry in the process.

**METHODS OF CALCULATING CMRR**

CMRR can be calculated from drill core or from roof rock exposures such as highwalls and overcasts. There are currently three methods which can be used to calculate CMRR from drill core.

**Standard Method**

This is the most comprehensive and time consuming method for calculating CMRR. The Geologist or Engineer inspects the core and divides it into units which have similar geotechnical properties such as uniaxial compressive strength, bedding and discontinuities. The properties of each unit are determined including fracture spacing, RQD, diametral point load testing, uniaxial compressive strength from rock testing or axial point load testing or sonic log, moisture sensitivity, weak unit contacts, and the presence of overlying weak rock. The units and their properties are then entered into the CMRR spreadsheet (provided with Colwell, 2003 or from NIOSH www.cdc.gov/niosh/mining/topics/groundcontrol/groundcontrol.htm) and the individual unit ratings and overall CMRR is calculated. Each CMRR calculation is done separately. The CMRR data is collected and entered manually. This is the best method to use when drill core needs to be assessed on a relatively infrequent basis, or only a small number calculations need to be made at one time. The calculation can only be done on a maximum of five separate units which tends to limit the CMRR calculation to a 2 m length of core.

**Rapid Rating**

This is the quickest and most efficient method of calculating CMRR values on large quantities of drilling data. The geological data, comprising the digital Lithology log fracture log, geophysical logs, core photos, and rock testing results is imported into the Rapid Rating Program. The Rapid Rating program analyses the data to automatically create geotechnical unit and calculate their properties, the unit ratings and the CMRR. The logic and calculations used by Rapid Rating are exactly the same as those in the Colwell spreadsheet (Colwell 2003). The CMRR data used is the normal geological data collected for each drill core so the core does not need to be inspected or tested separately to calculate CMRR. The data for many holes is imported and CMRR is calculated at the same time, and without separate calculation. Calculating CMRR on a set of geological data using Rapid Rating requires a standard setup time for each data set, and the processing time is similar whether there are 10 or 20 holes. Each CMRR can in theory have an infinite number of units and an infinite length although there are practical/geotechnical limitations to the number of units and CMRR length which can be used.

**Hole Log Unit Rating**

This is the quickest method of calculating unit ratings (rather than CMRRs – as the unit ratings have to be combined to create a CMRR). A similar approach is used as described above in rapid rating, except that the unit ratings are done on lithological units rather than by identifying discrete geotechnical units. These ratings can be calculated more quickly than the Rapid Rating CMRR and are useful to include on a hole composite log to
highlight variations in rock mass strength between lithological units. They cannot be used to calculate full
CMRRs and they cannot be compared with other full CMRR results or in the CMRR design tools such as ALTSII
or other empirical design approaches which use CMRR values.

RAPID RATING METHODOLOGY

Rapid Rating automates the process of:

- identifying lithological and structural units,
- identifying moisture sensitive units,
- calculating fracture spacing within units,
- calculating average UCS for units,
- identifying weak contacts,
- calculating strong bed adjustments,
- calculating unit ratings, and
- calculating CMRR values.

The Rapid Rating system calculates CMRR as described below:

1. Create UCS Units – Sonic data is imported into the program. Inferred UCS is calculated for each
   sonic depth. The difference between each UCS value is calculated. The program then analyses
   the data and identifies where gradient changes in the UCS plot occur and where steep gradients
   occur. The program identifies patterns in the gradients and picks out bumps. A UCS unit is
   defined by the start and end of the bump. The program identifies plateau changes, where the UCS
   steps up or down. In this case the UCS unit is split in the middle of the change section. (See Figure 1).

   ![Fig 1: UCS units overlaid on UCS plot.](image)

2. Create Lithology Units – Lithology data is imported into the program. The lithology dictionary is
   used to convert the data to text. CMRR lithology units are created with a lithological description
   and are given a moisture sensitivity deduction.

3. Create CMRR units – The program compiles the UCS units and the lithological units. If there are
   a number of UCS units within one lithological unit, the lithological unit is split up by the UCS
   units and vice versa. If a UCS unit start or end depth is similar but not exactly the same as a
   lithological unit, the CMRR will use the UCS unit depth. If a CMRR unit encompasses more than
   one lithological unit, the description used will be that of the largest lithological component. Units
   which are 5 cm thick or less are removed and joined to adjacent units. Units which are 15 cm
   thick or less are removed and joined to adjacent units if the UCS difference is not significant. (See
   Figure 2).

4. Calculate Average UCS for each CMRR Unit – The program collects the individual UCS values
   over the length of each CMRR unit and averages them.
5. Calculate Fracture Spacing, RQD, Diametral Strength – Fracture spacing, RQD and Diametral data is imported into the program. An average depth is calculated for each fracture by averaging the ‘from’ and ‘to’ depths. The fracture spacing is calculated at each fracture as the distance between it and the previous fracture. The fracture spacing for each CMRR unit is calculated by averaging the fracture spacings which occur within the CMRR unit. For example the fracture spacing for the unit depicted below would be \( \frac{FS1 + FS2 + FS3 + FS4}{4} \). (See Figure 3).

6. The program calculates the UCS rating, Discontinuity Rating and the Unit Rating.
7. The program identifies the location of weak bedding planes where lithological changes occur and one of the two lithologies has some form of potentially “slippery” mineralogy component such as coal, clay, tuff, claystone, mudstone etc.
8. The depths and horizons which CMRR results are required for the input. If groundwater is present, the groundwater adjustment is input. The data is reviewed and surcharge is allocated against the effected horizons.

**Fig 2:** Lithological and UCS units are combined to create CMRR units.

**Fig 3:** Fracture spacing is the average of the spacings between fractures which occur in the CMRR Unit.
9. The program calculates the Strong Bed adjustment, weak contacts adjustment and the thickness weighted average rating for each depth and horizon. Then it adds the adjustments, the rating, the groundwater adjustment and the surcharge adjustment to create a CMRR result.

**GEOTECHNICAL ASSESSMENT AND DESIGN WITH CMRR**

The CMRR is best used as part of a holistic approach to geotechnical characterisation and mine design (see Figure 4). By focusing on collecting and analysing a base level of essential geotechnical data an accurate and reliable set of rock properties can be established. This includes rock testing analysis, fracture data, geophysical log analysis, acoustic scanner structural analysis, borehole breakout stress mapping and composite log analysis. Quality base data will reduce the risks of inappropriate design to operational implementation.

**Fig 4: Geotechnical assessment and design with CMRR.**
Quality geotechnical data and analysis can then be used to conduct geotechnical characterisation, that is, to calculate CMRR, identify geotechnical domains, conduct geotechnical sensitivity analysis and create hazard plans.

Robust geotechnical characterisation can be utilised in a comprehensive design approach which may include numerical modelling, empirical design and benchmarking, analytical design, and observational analysis. It is preferable to use at least two design approaches together, if possible, as each design tool has its own limitations.

Design is undertaken using a risk based approach which allows the operator to consider a range of potential operational approaches to manage the geotechnical risks whilst taking into account the implications of different approaches.

**BENEFITS OF USING CMRR**

There are many benefits of using CMRR:

- It allows numerous complex rock properties to be combined into one quantitative rock mass strength indicator in a consistent manner.
- It allows site geology to be displayed simply on a single plan using CMRR contours rather than requiring a number of plans such as uniaxial compressive strength, lithology, rock quality designation (RQD) to be overlaid.
- It allows better communication of geotechnical conditions between geologists and engineers and the workforce in general.
- It can be used to highlight areas of rock mass strength variability and to identify geotechnical domains.
- It can be used to compare ground control experiences between sites and allow benchmarking and empirical design.
- In conjunction with other geotechnical data, it can be used to indicate the potential for extended cuts.
- It can be used in ALTSII (Colwell, Hill, Frith, 2003) in conjunction with other geotechnical data to indicate tailgate standing support requirements.
- It can be used in conjunction with other geotechnical data to indicate primary support requirements.
- It can be used in conjunction with other geotechnical data in Strata Control Management Plans and in trigger response action plans.

**RAPID RATING BENEFITS**

- Results are generated much faster than the standard method.
- CMRR can be calculated over any horizon and with an unlimited number of units. CMRR can be calculated on numerous bolt lengths and horizons for sensitivity analysis.
- Improved repeatability of a CMRR calculation and reduced human errors.
- CMRR can be calculated from standard exploration data and does not require re-logging.
- Data is imported digitally, not manually entered, reducing double handling and errors.
- The Rapid Rating Program identifies discrete geotechnical units, calculates unit ratings and CMRR values automatically.
- Recalculations can be done quickly and easily.

**RAPID RATING LIMITATIONS**

The Rapid Rating system is designed to handle large quantities of data, and is not as cost effective to use on a small number of holes (eg. to calculate one 2 m CMRR).

Currently Rapid Rating is not a commercially available software package, which means that calculations using Rapid Rating can only be done by SCT Operations Pty Ltd. It would be quite possible to build a user friendly commercial software package if the need is present in the industry.

Diametral point load testing is not routinely conducted with coring. The benefit of Rapid Rating is that it is quicker and cheaper than the standard method and this is partly due to the fact that the core does not need to be re-
logged and retested. Unfortunately this means that CMRR generally has to be calculated by Rapid Rating without diametral point load testing data. There are some circumstances where the core is weakly bedded but unfractured. In these situations the fracture spacing and RQD ratings can be quite high whereas the diametral PLT would be low and so in its absence, may lead to a high CMRR which is unrepresentative. It may be possible in future to correlate diametral point load results with bedding (identified visually or in geophysical logging) to enable this error to be reduced. Currently, the results are ground truthed and qualified if there is a possibility of the error being present in a result.

Rapid Rating makes it possible to calculate CMRR over any core length (e.g. 0.5 m, 1 m, 2 m, 4 m, 8 m or 100 m) as the number of units is not limited in the calculation process. However, CMRR is an empirical system and was specifically developed to be used over a length of around 2 m. As such the weightings, equations and methodology can not be assumed to be useful for other lengths. It may be useful to calculate CMRR on the 0.5m horizon to indicate the potential need for roof skin control such as mesh, or to indicate the extent of longwall dilution for a given roof horizon. It may be useful to calculate CMRR on the 4 m and 8 m horizons to indicate the potential need for long tendon support. So whilst it may be interesting to calculate CMRRs on other horizons and start to build a database for these horizons, until such time as significant research is conducted into the validity of applying CMRR to the other horizons, and until substantial empirical databases are developed for the other horizons the results should be used with extreme caution.

FUTURE DEVELOPMENTS

If a correlation between diametral point load tests and visible bedding, geophysical logs and fracture spacing can be developed then the limitations on calculating CMRR retrospectively and using exploration data which was not obtained with CMRR in mind can be significantly reduced. This has enormous potential to decrease the cost of developing a CMRR database for a site and potential to significantly increase the amount of CMRR data which can be generated.

Similarly, if a reasonable indication of discontinuity rating can be developed by correlating fracture spacing and diametral point load tests with geophysical logs (even if only on a site to site basis) the possibility of determining CMRR from non core holes may be realised. This would have an even larger impact on the cost of developing an extensive CMRR database and amount of CMRR data which can be generated.

There are possibly valuable applications of the development of CMRR databases of other horizons such as 3-8m roof horizon to indicate the need for long tendon reinforcement, the 0.2-0.5 m horizon to indicate roof skin conditions and longwall dilution, and the floor horizon (with adjustments to ratings and methodology) to indicate floor conditions.

CONCLUSIONS

The CMRR is an extremely valuable geotechnical characterisation tool which can significantly simplify and enhance the identification and communication of different geotechnical regimes. Rapid Rating is a new method of calculating CMRR from exploration drilling data which significantly reduces the cost and time to calculate CMRR on large quantities of data. It increases the flexibility in CMRR calculations and reduces human error as well as ensuring a consistent approach in identifying units and their properties.

As CMRR becomes more widely adopted in the Australian mining industry it is important for mining professionals to remember that it is a rock mass quality (strength) indicator and not a roof stability indicator. As such it needs to be considered in combination with all of the other geotechnical design factors such as stress, geometry, roadway use, structure. It should be used in combination with other design approaches to determine mine design parameters such as support patterns.

The benefits to the industry from improving the CMRR will increase if the results can be made simpler to obtain more geological information such as pre-existing drilling data and non core holes can be used. This will allow larger site CMRR databases to be created with a higher spatial density of geotechnical information and lead to increased safety and reduced operational costs through more appropriate geotechnical design.
ACKNOWLEDGEMENTS

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REFERENCES

WHY UNIAXIAL COMpressive STRENGTH AND YOUNG’S MODULUS ARE COMMONLY POOR INDICATORS OF ROADWAY ROOF STABILITY – EXCEPT IN THE TAILGATE

Mark Colwell¹ and Russell Frith²

ABSTRACT: For many years underground rock mechanics and in particular, roadway/tunnel roof stability has been underpinned by the often unchallenged assumption that roof strength (as defined by the UCS) and stiffness (E) are key stability controls. This has logically led to the proliferation of laboratory testing of rock specimens and the development of indirect geophysical methods to gain estimates of these two rock parameters. Furthermore, many design methods are significantly focussed on replicating rock mass behaviour through either intact or failed constitutive models. Demonstrably the strength and stiffness of the host rock material is commonly used as one of the key indicators of excavation roof stability and it finds either direct or indirect use in just about every rock mass rating system in use today.

In more recent times there has been a common move to consider and apply (even if only conceptually at the current time) structural engineering type principles (eg, buckling) to coal mine roadway roof (and rib) stability. Similarly our knowledge of the in situ stress environment and its likely origins has improved significantly, largely through stress measurements and subsequent analysis. This paper combines knowledge in both of these fundamental areas through a deterministic model for roadway roof stability and in combination with field examples, reaches the almost certainly controversial conclusion that UCS and E are commonly irrelevant, albeit that the former may provide an indication of other relevant geotechnical parameters (eg, bedding cohesion).

As with all hypotheses or rules, there are naturally exceptions and in this case, the most obvious is the tailgate of the longwall panel (with adjacent goaf). Due to the significant change in the strata loading environment of a longwall tailgate as compared to first workings for example, the stability equation materially changes so that UCS and E become critical controls.

The point of the paper is to present a different perspective on a traditional mining problem and to challenge geotechnical professionals to keep thinking “outside of the square” in the never-ending endeavour to improve our understanding of the engineering problems we regularly face. Such an understanding impacts upon such issues as geotechnical data collection from borecore, support hardware requirements and design capabilities. Therefore making the assumption that our understanding is always fundamentally correct could in fact be limiting the development of new and improved engineering.

INTRODUCTION

Material strength is a convenient engineering property. The statement that something is “strong” conjures up certain images and conversely something that is “weak” is readily understood by all. Furthermore material strength is a relatively straightforward material parameter to ascertain through laboratory testing. Therefore it is understandable that in rock mechanics and strata control, the terms “strong roof” and “weak roof” proliferate. Major research projects have been undertaken (SCT 2000) simply focusing on weak strata on the assumption that it is somehow a different genre to “strong” roof and is perhaps governed by a totally different set of constitutive laws and controls.

As a fundamental tenet, the load-bearing ability of any engineered structure is always related to the external and internal loads acting. A structural engineer would never state that a structure is “strong” simply because it is made out of high grade steel for example and conversely, an earth bank can accommodate very high applied loads, even though it is made from materials that are “weak” in comparison. To generalise on the stability of an engineering structure based solely on material strength is clearly inappropriate.

This paper explores the hypothesis that a significant portion of the in situ stress applied to the roof of a mine roadway is directly related to the strength of the rock material that it is contained within. Therefore on the basis

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that the ability of the roof to accommodate an externally applied stress is also related in some way to its material strength, leads to the inevitable conclusion that its overall stability or instability (as defined by a Factor of Safety measure) should have a tenuous link to the material strength involved. If true for the roof of coal mine roadways, there should be ample evidence available to support this hypothesis.

Demonstrating such an outcome has significant potential ramifications to both geotechnical analysis and future strata control research. In terms of geotechnical studies it would surely prompt a re-assessment of the basis of numerical codes, which are highly reliant upon laboratory strength test data and typically make broad assumptions regarding the general magnitude of the in situ stresses.

At the current time there is also a significant move underway to further classify strata conditions from down-the-hole geophysical data (eg, sonic velocity, gamma etc – Medhurst and Hatherly, 2005). This is underpinned by the well known link between sonic velocity and UCS (albeit site specific calibration linked back to laboratory derived values is generally required to provide credible guidance on local material strengths), the intent also being to try to link such geophysical data from boreholes to a rock mass rating system such as the Coal Mine Roof Rating. This would indeed be a quantum step forward in rock mass characterisation, but it is vital that such a process does not inadvertently overlook any of the critical rock mass parameters, which may not always include material strength. This paper is being written to provoke further thought and discussion as to how rock masses in underground coal mining need to be classified and what other pieces of information are vital when undertaking a credible geotechnical assessment.

**GENERAL OVERVIEW**

In order to evaluate the stability of an unbolted mine roof few would probably disagree that the essential requirement is one of comparing the applied ground stresses against the ability of the rock mass to accommodate such stresses (termed “competence” herein to differentiate from material strength).

Using the analogy of coal pillar stability and design, a Factor of Safety argument or stability measure can be applied to the natural stability or self-supporting ability of a coal mine roof along the lines of:

\[ \text{stability} = \frac{\text{roof competence}}{\text{applied stress}} \]  

...(1)

Note that equation (1) is a simplified version of the equation that also includes the role of ground support, namely:

\[ \text{stability} = \frac{\text{roof competence} + \text{ground support}}{\text{applied stress}} \]

The role of ground support is not being considered by this paper, hence the removal of the term from the stability equation.

Unlike current coal pillar design, the assignment of credible values for both roof competence and applied stress is not generally agreed upon by the strata control fraternity. There is no roof stability equivalent to the fundamental work of either Salamon or Bieniawski that, in the aftermath of the Coalbrook disaster in South Africa, set the framework for the current understanding and design ability in the stability of coal pillars.

Yet the fundamental nature of the problem in the roof of a mine roadway is not materially different. Stresses are applied to the roof structure and according to it’s makeup, it will either be stable or unstable. The technique of cut and flit roadway development either lives or dies by this basic issue. What is less straightforward is a means by which credible numerical values can be applied to the key parameters and consideration of this leads to the suggestion stated in the title of the paper; that UCS and E may not be quite as important to roof stability as has perhaps been assumed in the past.

**A MODEL FOR HORIZONTAL STRESS IN COAL MEASURES STRATA**

When putting any explanatory model forward, its validity may be no greater than the measured data on which it is based and even if it proves to have more widespread application, the existence of data that disproves the theory is always a possibility. Nevertheless any model, even if limited in its application, is better than none at all as others will invariably refine and improve it based on their own data and knowledge. It is with this limitation that the model for horizontal stress is described herein.
The model is not new and has been published by others (Nemcik et al. 2005), the focus here being on demonstrating the validity of the model as an input into the roof stability equation.

The model uses the assumption that there are two primary sources of horizontal stress in the ground, one being the vertical stress through Poisson’s Ratio or $K_o$ effects, the other being tectonic strain induced as a result of large-scale plate movements. Therefore:

$$\sigma_H = \sigma_v \left(\frac{\nu}{1-\nu}\right) + E \varepsilon \quad \ldots(2)$$

$$\sigma_h = f(\sigma_H) \quad \ldots(3)$$

$$\sigma_v = \rho \cdot g \cdot h \quad \ldots(4)$$

where:
- $\sigma_H$ = major horizontal stress
- $\nu$ = Poisson’s Ratio
- $E$ = Young’s Modulus
- $\varepsilon$ = tectonic strain (also referred to as the “Tectonic Stress Factor” by Nemcik et al. 2005)
- $\sigma_h$ = minor horizontal stress
- $\sigma_v$ = vertical stress as given by weight of overburden considerations
- $\left(\frac{\nu}{1-\nu}\right)$ = numerical determination of $K_o$

It is noted that the potential for a residual horizontal stress in the ground (emanating from Poisson’s Ratio effects with much higher depths of cover that has been removed via erosion over geological time), is not considered herein as it is outside the scope of the paper. Suffice to state that it is acknowledged as a potential source of horizontal stress and in some coalfields (e.g. Southern Coalfield of NSW) significant magnitudes can be reliably inferred from the analysis of in situ stress measurement data. However it will not be considered further by this paper, accepting that it is a relevant consideration in some geotechnical environments.

Figures 1 and 2 show the results of a basic analysis of stress measurement data from an Australian longwall mine, the measured horizontal stresses having been adjusted for depth of cover and $K_o$ effects so that tectonic horizontal stress components can be directly evaluated. Note that in all cases, the curve fits used have not been forced to go through the origin.

**Fig. 1** - Measured major horizontal stress (with $K_o$ component removed) versus Young’s Modulus of host rock

$y = 0.4664x - 0.0548$

$R^2 = 0.6908$
Based on the outcomes shown in Figures 1 and 2, the following are evident:

- As suggested by equation (2), the tectonic component of the major horizontal stress is strongly if not uniquely linked to the Young’s Modulus or stiffness of the host rock material.
- The tectonic component of the minor horizontal stress is strongly linked to that of the major horizontal stress. It is interesting to note that if the gradient (0.5) of the curve fit in Figure 2 is taken to be an \textit{in situ} estimate of $K_o$ (as stated in the first component of equation (2)), a back-calculated value for Poisson’s Ratio of around 0.33 is found, this not being outside the credible limits for coal measures strata.

The point to be made is that a significant proportion of the horizontal stress in the ground is often (although not always – e.g. a coal deposit adjacent to steep topography) directly linked to the Young’s Modulus or stiffness of the host material. This is a critical principle for the remainder of the paper.

\textbf{IS A STIFF ROCK TYPE NECESSARILY A STRONG ROCK TYPE?}

Accepting that in general terms, rocks with a higher Young’s Modulus contain a higher level of tectonic horizontal stress (all other factors being equal), the next logical question to ask is whether stiff rocks are also strong rocks.

Figure 3 shows laboratory rock testing results from a mining project in Australia, in terms of the relationship between Young’s Modulus and UCS.

It is clearly evident from the results and curve fit shown in Figure 3 that the UCS and Young’s Modulus are strongly linked, albeit that there is some scatter in the data set. Nonetheless statistically the two parameters are linked with a high confidence level and data sets from other mining projects show exactly the same relationship, with surprisingly similar correlations.

As a result, it can be stated with confidence that as a general rule, stiff rocks are also strong rocks. When this is combined with the finding of the previous section, it is also true to say that rock types containing higher levels of horizontal stress are also the stronger rock types.

Referring to equation (1) and taking the simplistic view that in some way the competence of a rock mass is a function of its material strength, it is evident that the UCS (and hence Young’s Modulus as the two are generally
interchangeable) is potentially a major contributing factor to both the numerator and the denominator (which for roadway development relates to the *in situ* horizontal stress). Therefore UCS or E effects largely cancel out of the equation and leads to the conclusion that the roof stability “Factor of Safety” may not always be strongly linked to the UCS or modulus of the host material.

![Graph](image)

Fig. 3 - UCS versus Young’s Modulus relationship as found from laboratory testing data

Before this concept is taken any further, it is necessary to examine whether it holds true when tested by reference to a more comprehensive model of roadway roof stability, this being that presented by Frith 2000 when examining the issue of cribless TG’s.

**A GENERAL MODEL FOR ROADWAY ROOF BEHAVIOUR IN A HORIZONTALLY LAYERED STRATA SEQUENCE**

A fundamental issue to consider in roadway roof stability is the mode of roof behaviour occurring as the roadway is being formed and/or during subsequent mining activities. This will have a wide ranging effect (varying from none to highly significant) on such issues as the self-supporting ability of the rock itself, bolting requirements, timing of support installation and ultimately the potential for roof instability.

There are two primary modes of roof behaviour (STATIC and BUCKLING) which have been identified and generally proven through extensive monitoring studies at a large number of mines in Australia. Both can lead to stable roof conditions, but both have one or several associated roof failure modes which can potentially lead to a roof fall situation if not adequately controlled.

The two basic modes of roof behaviour will now be described.

**Static roof:** this involves roof conditions whereby the level of horizontal stress across the roof is insufficient to cause bedding plane separation, which thus prevents the roof measures breaking down into thinner discrete units. Essentially, the roof measures “absorb” the stress changes due to roadway formation without undergoing any change in state apart from primarily elastic movement.

The lower the horizontal stress across the roof, the more likely that static roof will persist. Similarly in general terms, increased bedding plane cohesion should also increase the likelihood of static roof conditions being maintained as this is the primary rock parameter that acts to prevent bedding plane failure and separation.
In terms of extremes, a highly stressed roof environment at 500m depth of cover can exhibit static behaviour in combination with thickly bedded or massive roof measures. In contrast, lower horizontal stresses can cause buckling type behaviour in a thinly bedded roof environment. The point is that both the stresses and the nature of the roof must be considered in combination when assessing the likely mode of roof behaviour, as also covered in equation (1).

Typically, a static roof environment will undergo < 5 mm of roof movement as a result of roadway formation and in some instances, no discernible roof movement can be detected by roof extensometry. It is the most stable roof condition and is typically self-supporting, it being the fundamental requirement for stable extended cuts during development as will be discussed later.

Figure 4 illustrates a static roof schematically and gives an example of associated extensometry data.

**Buckling roof:** buckling roof behaviour occurs once a portion of the roof measures undergo tensile and/or shear bedding plane failure resulting in the formation of a number of thinner discrete units (“columns”) acting under the action of horizontal stress. For the purpose of this paper, this behaviour will be termed as buckling, recognising that it is not a strictly correct use of the term.
The mode of deflection of the roof measures changes with the onset of buckling from primarily elastic expansion in a static roof to downwards buckling of the roof measures. This causes a large increase in the magnitude of roof displacement for any given horizontal stress due to a reduction in the overall stiffness of the various thin independent strata units.

The main point of relevance herein in relation to a buckling roof environment is that it is not necessarily self-supporting and generally relies upon the application of specific ground support to ensure stability is maintained. Unlike a static roof environment, the occurrence of a buckling roof would be expected to be highly detrimental to the stability of extended cuts during roadway development, to the point that it commonly necessitates the use of a miner/bolter installing roof support in sequence close to the development face.

Figure 5 illustrates the occurrence of a buckling roof schematically and presents typical time-dependent displacement trends in the roof leading to an equilibrium condition being attained.

* tensile bedding failure occurs
* roof measures sub-divide into thinner discrete units
* buckling roof - high displacements

Fig. 5 - Schematic illustration of buckling roof behaviour and associated extensometry data

As a point of interest, Figures 6, 7 and 8 show extensometer data examples of what are taken to be buckling roof environments in Australia, the US and the UK, the similarity in their form being self-evident.
ANALYSIS OF THE US DATABASE ON THE STABILITY OF EXTENDED CUTS

The US database on the stability of extended cuts during development is an invaluable assessment tool, as it is one of the few roadway or tunnel roof stability databases that does not include the effect of installed support. It is as good a representation of equation (1) as can be found and the provision of information contained within the database by Dr Chris Mark of NIOSH is duly acknowledged.

The database classifies the stability of extended cuts at a number of US coal mines according to whether they were “always stable”, “sometimes stable” or “never stable”. In addition to these mining outcomes, the database also includes many of the basic geotechnical parameters of interest, including depth of cover, roadway width and the Coal Mine Roof Rating (including the individual CMRR parameter ratings) – Mark 1998.

Combining equations (1) and (2) with the hypothesis that for the occurrence of either a static or buckling roof condition, bedding plane cohesion is the key rock mass parameter, the following is apparent:

\[
\text{stability} = f(\text{bedding cohesion})
\]

For equation (5) to be generally true, the following statements should in theory be supported by the contents of the US extended cut database:

(a) There should be some form of relationship between bedding plane cohesion within the immediate roof of the roadway and the depth of cover, stable cuts requiring higher cohesion levels at higher depths of cover for “always stable” outcomes.

(b) If cohesion and UCS are dependent variables (along the lines of that shown in Figure 3 for UCS and E), a poor correlation with stability outcomes should be found when the two are plotted against each other. However, if they are independent variables or there is significant scatter in the relationship between the two, some correlation with stability outcomes may be evident in the same plot.
Fig. 7 - Roof extensometry data from the US suggesting the occurrence of a “Buckling” roof environment (Oyler et al 2005)

Fig. 8 - Roof extensometry data from the UK suggesting the occurrence of a “Buckling” roof environment (Adams 2003)
Figure 9 shows (for all of the single strata unit cases within the database) the bedding plane cohesion rating against depth of cover, the cases being sub-divided into the three stability outcomes. From this figure it is evident, at least in terms of general trends, that:

- for any given depth of cover (especially up to 300 m which covers the majority of the case histories), the most stable outcomes relate to the highest levels of bedding cohesion, and
- as the depth of cover increases, so does the bedding plane cohesion associated with each of the three stability cases.

Therefore it would seem, as implied by equation (5), that there is some correlation between depth of cover, bedding plane cohesion and the stability of extended cuts.

Figure 10 shows the bedding plane cohesion rating plotted against the material strength rating for each of the single roof unit cases, as well as the cut stability outcome in each particular case.

The following outcomes are apparent from the data contained within Figure 10:

- Whilst there is a general trend for bedding plane cohesion to increase in line with material strength (as shown by the dotted line), there is a significant degree of scatter. This is not surprising as bedding planes often comprise different material (e.g. mica, carbonaceous material) as compared to the host rock, therefore a significant scatter would be expected.
- Accepting that the Strength Rating is also a possible indicator of the tectonic component of horizontal stress acting, the “never stable” cases are all associated with weaker levels of bedding cohesion, as compared to the “always stable” cases which tend towards stronger cohesion. As would be expected, the “sometimes stable” cases are located in between with overlap into both the “always stable” and “never stable” populations.

The data set is perhaps not comprehensive enough to be absolutely definitive on this issue, but the apparent trends certainly support the suggestion that the stability of extended cuts is a function of both bedding plane cohesion and material strength (i.e. UCS).
Overall the general trends found within the US database on extended cut stability lead to the conclusion that material strength (i.e. UCS or E) in isolation does not allow a reliable prediction of cut stability to be made. This can also be clearly seen in Figure 10 whereby the stable cases cover the full range of material strength ratings from 1 to 5. Therefore, other factors also need due consideration including bedding plane cohesion and depth of cover as a minimum.

It is interesting to note that the relative importance of the material strength rating within the Coal Mine Roof Rating has down-graded on at least one occasion. This is also perhaps evidence of the relative insignificance of UCS and E to the overall roof stability equation, although as will be discussed later there are some notable exceptions whereby it becomes a critical stability parameter.

**BUCKLING THEORY AND THE SIGNIFICANCE OF MATERIAL STRENGTH**

The behaviour of thin columns under load is covered by a number of theoretical treatments that, in combination, can be used to provide an estimate of load-bearing capacity across a full range of column geometry. For the purposes of this paper, use will be made of Euler Buckling theory to demonstrate key principles.

Euler Buckling theory defines the critical buckling stress ($\sigma_{cr}$) of a thin column (i.e. the stress at which uncontrolled buckling and structural failure will initiate) as follows:

$$\sigma_{cr} = \frac{\pi^2 E}{(L_e/r)^2}$$  \hspace{1cm} \text{(7)}$$

where: $E$ = Young’s Modulus  
$L_e/r$ = Slenderness Ratio = f(column length, thickness)  
$L_e$ = effective length of the column  
r = radius of gyration

It is noted that Euler Buckling theory only applies to a certain range of Slenderness Ratios and does not define the complete behaviour of thin columns. It is being used for illustrative purposes only.
Therefore it is clear that the maximum load-bearing capacity of a thin buckling column is a direct function of both its Young’s Modulus and dimensions. However it was shown earlier that Young’s Modulus can be replaced with UCS, so that it is also true that the maximum load-bearing capacity of a thin buckling column is directly related to the strength of the host material.

The above basic analysis demonstrates a fundamental tenet of structural analysis, namely that the maximum load-bearing capacity of a structure is determined as a proportion of the material strength of the structure, the proportion being related to its geometry.

When this finding is substituted into equation (1) it can be shown that for a buckling roof in a predominantly tectonic horizontal stress environment, stability has almost no link to material strength, but more to the problem geometry (i.e. column length and thickness).

For the roadway roof stability problem these two parameters are given by roadway width and bedding thicknesses respectively. Few geotechnical engineers would disagree that in aggressive conditions, roadway or tunnel roof stability decreases in line with increasing roof span and similarly, the roof becomes less stable and more difficult to control as bedding thicknesses decrease in the host rock mass.

**GENERIC SUPPORTING EXAMPLES**

In order to complete the discussion, it is worth citing some generic examples that further confirm the suggestion that UCS and E are commonly poor indicators of roadway roof stability.

**Thick Coal Roof**

The most obvious example to consider is that of a thick coal roof. Mining experience dictates that the presence of a thick coal roof is commonly a more favourable environment for roadway development purposes, as compared to some of the rock sediments above. However coal is far from being the strongest of material when compared with many of the rock types commonly encountered.

Two geotechnical issues are relevant to coal as a development roof environment. Firstly due to its low strength it also has a low Young’s Modulus so that the tectonic component of horizontal stress is reduced as a direct result. The low strength of the material is directly compensated for by the low Young’s Modulus and its inability to attract high levels of tectonic horizontal stress.

The second issue is that bedding thicknesses within many coals are significantly greater than thinly bedded rock sediments such as shales and laminates. Therefore any buckling within the coal roof that may want to occur under the action of the \textit{in situ} horizontal stress will be better accommodated as compared to a thinly bedded rock roof.

In this regard it is also interesting to note that a number of Australian longwall mines in thick seam environments have found that not only does the leaving of a coal skin in the immediate roof decrease roof flaking and small pieces dropping out, but if a sufficient thickness of coal roof is left in place (typically in excess of 1 to 1.5 m), the global stability of the roof can also be improved.

**Seam Splits**

Within the Australian coal industry, it has been recognised that areas containing splits in the roof of the coal seam can be associated with far more difficult roof conditions, than areas whereby the seam is coalesced as a single unit.

One of the features that is commonly found when evaluating strata competence in seam split locations is that the frequency of bedding planes/fractures (in both the coal and immediate roof) in borecores increases significantly, as compared to areas remote from a seam split. However the variation in material strengths in and around seam splits can be marginal at best and nowhere near the same magnitude of change as compared to the bedding fractures.

Therefore, the most obvious link between the deterioration in roadway roof stability in proximity to a seam split and geotechnical parameters from local borecore commonly relates to fracture spacing within the measures, not reductions in material strength.
Thickly Bedded to Massive Strata
At the other end of the scale, some of the most competent roadway roof conditions encountered relate to the presence of thickly bedded or massive strata in the immediate roof. Even at depths of cover down to 500 m, the presence of a thickly bedded to massive immediate roof environment can be associated with very benign development roof conditions, allowing extended cuts to be used and minimal roof support densities.

Massive strata contains few if any bedding planes so that the mechanism for a buckling roof environment (i.e. bedding plane failure) is not present. In thickly bedded strata, even if bedding plane failure does take place, the resultant strata units are sufficiently thick to still have a considerable amount of self-supporting ability prior to the installation of roof support. Either way, the self-supporting ability of the roof measures remains high.

WHAT HAPPENS IN THE TAILGATE OF THE LONGWALL?

As with all theories and concepts, there will always be exceptions and in this particular case, whilst there are several possibilities (e.g. coal seams within hillsides, very weak roof whereby self-weight effects dominate the loading environment), the most obvious is in the tailgate of a longwall face with adjacent goaf.

Frith 2000 discussed the issue of cribless tailgates and presented the roof loading model shown in Figure 11.

The basis of the loading model is that:

(a) a significant proportion of the in situ horizontal stress acting across the roadway has been eliminated due to the presence of an adjacent goaf and its inevitable horizontal stress relieving ability

(b) the primary source of strata loading during TG loading is in the form of vertical stress, this driving increased spalling of the coal ribs (which can give rise to an increased roof span) and also the development of increased horizontal stress across the roof via Poisson’s Ratio of $K_o$ effect.

If this loading mechanism is correct, the increase in horizontal stress across the roof of the TG will be some function of Poisson’s Ratio or:

$$\text{horizontal stress increase} = f(K_o) = f(\nu/[1-\nu])$$ ...(8)

Fig. 11 - Schematic illustration of general TG loading conditions (from Frith 2000)

Poisson’s Ratio is not always captured as part of laboratory rock testing programs and is probably the most difficult parameter to determine accurately. However Figure 12 shows a trend relationship found between Poisson’s Ratio and Young’s Modulus for one particular mining project and the general trend amongst the inevitable data scatter is for Poisson’s Ratio to decrease as Young’s Modulus increases.
As a result equation (8) can also be written as:

\[
\text{horizontal stress increase} = f(K_o) = f\left(E^{-1}/[1-E^{-1}]\right) \quad \text{…(8)}
\]

Therefore as Young’s Modulus decreases, the value of \(K_o\) may actually increase, such that the value of horizontal stress being generated across the TG roof also increases. This is in direct contrast to the model for the \textit{in situ} horizontal stresses discussed earlier, which shows that Young’s Modulus and the tectonic component of the horizontal stress are directly rather than inversely proportional.

Returning to the general stability equation given by equation (1) and substituting in the specifics for a longwall tailgate and a buckling roof environment, the following is apparent:

\[
stability = f(E \text{ or UCS}) /
f(\text{depth, } 1/E \text{ or } 1/\text{UCS}) \quad \text{…(9)}
\]

Unlike the case of roadway development or indeed the MG end of the face whereby the \textit{in situ} horizontal stresses acting are of most significance to roadway roof stability, roof stability is now not independent of UCS or E, but directly related to UCS or E.

\[R^2 = 0.1996\]

Fig. 12 - Laboratory derived values for Young's Modulus and Poisson’s Ratio for a mining project

If the concepts described herein and equation (9) have any credibility, longwall mining experiences should show that for low strength or modulus roof material (in particular a thick coal roof), roadway roof stability can reduce significantly and rapidly as part of TG loading, whereas prior to this (i.e. development and MG loading), roof stability had been quite benign and of minimal concern.

The Australian coal industry contains a number of examples whereby gateroad roof stability relates to the presence of a thick coal roof. It would be misleading though to simply suggest that those mines developing gate roadways with a thick coal roof are typically associated with very high levels of roof instability during TG loading (ground support controls mitigate against this risk).

However in the general experience of the authors, despite the thick coal roof often providing relatively benign roof conditions during development and through to MG loading, such longwall mines tend to be associated with generally more aggressive roof conditions in the TG. In other words a clear link with roof conditions prior to the onset of TG loading may not be evident in a thick coal roof scenario.
Therefore if roadway roof conditions prior to TG Loading are not clearly indicative of likely future TG conditions, inadequate levels of secondary support can be installed, the inadequacy only becoming evident once difficulties are experienced in close proximity to the approaching longwall face.

In the experience of the authors, the above described scenario is most likely to occur in conjunction with the presence of a thick coal roof, as it is being protected by its low Young’s Modulus before the onset of TG Loading, but then de-stabilised by its low Young’s Modulus during TG Loading.

**SUMMARY**

The paper has developed and presented a series of arguments that lead to the conclusion that as a general rule, the material strength of a coal mine roadway roof in isolation is not a particularly good indicator of likely roof stability under the action of the *in situ* horizontal stress (i.e. development and the approach of the longwall at the MG end of the face). The primary issue is that in those environments whereby the *in situ* horizontal stress is significantly influenced by tectonics, highly stressed strata units will also be high strength units.

Nonetheless, it is common to hear mining personnel classifying future mining areas based on whether the immediate roof material has tested as being either “strong” or “weak”. In some cases, mine site exploration activities even dispense with the collection and rating of roof core, this being replaced with a roof strength index derived solely from the borehole sonic log.

Only in those cases whereby the UCS of the host material provided a reasonable indirect indication of both bedding plane cohesion and/or bedding thicknesses, would future roof stability during development be well linked to material UCS. It is beyond the scope of this paper to consider this in detail, but presumably would vary from site to site dependent upon local geotechnical conditions.

Another possible exception relates to the presence of very weak roof material (in the order of only a few MPa) in conjunction with very weak bedding cohesion. In this situation, despite the low Young’s Modulus and so low potential for tectonic horizontal stresses, it is possible that the self-weight of the roof material itself becomes the significant driver for bedding plane failure and hence, roof instability without installed roof support.

Demonstrably, the situation of a longwall tailgate with an adjacent goaf does not conform to the general findings for roadway development. Mining experiences fit with the theoretical treatment that suggests that the strength and stiffness of the host material is in fact a significant determining factor in TG roof stability during extraction.

The change in the strata loading environment that occurs between the MG end of the face and subsequent TG Loading is a material change in that it is driven by two totally different processes and has a significant impact upon roadway roof stability. As a result, the roof stability rules will also inevitably change and a conceptual appreciation of both is required if ground support practices are to be appropriately tailored.

The concepts presented in this paper lead to three basic questions that need to be posed for consideration by the strata control fraternity:

1. Are we in danger of missing out on vital geotechnical information if we dispense with the collection and analysis of borecore in favour of indirect down-the-hole geophysical methods?
2. If UCS and E are relevant to roof stability in some scenarios, yet not in others, is there a case for removing it from the Coal Mine Roof Rating altogether or modifying the makeup of the Coal Mine Roof Rating according to the problem in question?
3. If UCS and E are perhaps second order considerations in roadway roof stability as compared say to bedding properties and thicknesses, are our numerical models unbalanced in terms of their relative ability to representatively incorporate the properties of the host material as compared to bedding and other discontinuities?

The answers to these questions are far from certain at the current time, but are critical in the on-going development of methods of geotechnical characterisation and design for underground coal mining purposes. They constitute a significant focus of on-going industry funded research and collaboration between the authors in their efforts to continually improve the geotechnical design tools available to the coal industry.
REFERENCES


JOINT STRUCTURE AND COAL STRENGTH AS CONTROLS ON RIB STABILITY

Ross Seedsman

ABSTRACT: The assessment of the stability of coal ribs needs to consider not only the impact of the cleating and jointing in the coal but also the possible onset of failure of the coal itself. The same planar, wedge and toppling failure modes seen in pitwall slopes can be present in underground roadways. A 20° offset of the roadway from the strike of the through-going joints reduces the fall hazard, but a 35° offset may be required to reduce delays in installing support. Mining-induced fractures (MIF) in coal may be a manifestation of brittle rock behaviour and its preferential location at the top and bottom of the rib and its continuing development outbye of the face can be explained by a dominantly vertical stress field within the coal.

INTRODUCTION

Ribs can present as great a hazard to the underground workforce as the roof. Until recently, most of the focus of mine operations and geotechnical researchers was on improving roof stability. A range of roof support hardware has been developed, bolter miners are used to bolt as close to the face as possible, and the alignment of roadways with respect to the horizontal stress field is now a fundamental principal of mine layout planning. By contrast, ribs have received lesser attention. In a 2001 feasibility study in which the author was involved, the discussion on roof support was 11 times longer than the discussion on rib support, and this is probably a good reflection of the relative state of the art.

Colwell (2005) highlighted the paucity of research into rib behaviour in Australian underground coal mines and went on to develop a comprehensive empirical design method for rib support. The 2001 feasibility study referred to above prompted the author to review the available literature on rib instability, with a particular focus on analytical methods. The analogy to pitwall instability had already been made, and recent work on brittle rock failure in hard rock mines (Martin et al, 1999) appeared to provide an explanation for the mining induced fractures (MIF) that had been identified by O’Beirne at al (1987).

This paper canvasses the range of rib failure modes that have been encountered by the author over the last 10 years and outlines how feasibility studies and mine operations can better incorporate rib stability considerations into the mine layouts. It provides an analytical framework to the empirical work of Colwell (2005).

RIB INSTABILITY

The primary hazard presented by ribs is related to large blocks of coal falling under gravity from the side of the roadway. There is a secondary hazard related to rib collapse leading to an increase in roof span, but it is considered that this is very much a lesser issue. Small spalling of coal ribs into small blocks does not represent a hazard unless it undercuts blocky coal higher in the rib.

There is a range of ways in which large blocks can be defined. Coal seams are characterized by the presence of discontinuities, referred to as cleats or joints. Strictly speaking, the term cleat is better restricted to the small scale fractures in the bands of bright coal; the through-going features of interest to the geotechnical engineer are joints. Depending on the dip, orientation and spacing of the coal joints with respect to the roadway driveage, toppling slabs (Figure 1a), planar slides (Figure 1b) and wedges can be defined. Depending on the thickness of the plates defined by the cleat, thin slabs can also undergo buckling, leading to the formation of detached blocks that can then collapse – this is the origin of the irregular rib line in Figure 1a.

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In other cases, where the roadway is aligned away from the strike of the joints, the top and bottom of the rib can deteriorate (Figure 2). If not supported, this fracturing can eventually define thin slabs of coal that can slide or topple into the roadway (Figure 3a). The surface that is developed parallel to the roadway, and hence defines the slab, is rough and irregular and appears vastly different from the natural joints (Figure 3b).
(a) Slab formed outbye of the cutting face and defined by a mining induced fracture parallel to roadway

(b) Intersection of mining induced fracture on left and coal joint on right

Fig. 3 - Mining induced fracturing

As well as the primary issue of supporting these blocks, there is also a need to optimize the installation time and quality of the support used. An irregular rib line can cause delays in the collaring of drill holes, and any open joints surfaces or broken coal can lead to the collapse of the holes on withdrawal of the drill rod or loss of resin and less than full encapsulation.

MECHANISMS AND SUPPORT

Structure control

If joints are present at the necessary alignment and spacing, the slide, wedge, toppling and buckling modes can develop at all depths and large blocks of coal mobilised. It is important not to ignore the impact of the dip of the coal seam. With coal joints being dominantly vertical, with perhaps a natural variation of +/−10° a cross dip of even 2-3° can add a significant differentiation between the up-dip and down-dip rib. The blocky nature of the rib may allow the use of spot bolts.

Rock slope stability text books present details of planar, wedge and toppling modes. There is readily available software for planar slides and wedges to simplify the calculation of the bolting support (www.rocscience.com). Sagaseta et al (2001) provide a simple method for the assessment of toppling and the necessary ground support. Support densities of less than about 3 tonne/m run of rib are typically obtained from the application of these tools. For buckling, a standard Euler analysis provides a good appreciation of the problem with thicker slabs buckling as the depth of cover increases (Figure 4). In all of these modes, there is a need to consider the need to provide skin restraint between the bolts and the presence of soft clay bands that may define smaller blocks.
The three observations that must be made with respect to the analogy to rock slope stability are as follows:

Firstly, standard practice to “remove” the planar slide hazard is to recommend a face alignment of more than 20° with respect to the joints in order. This certainly applies to the underground situation but it may not be sufficient. Based on recent observations in a Bowen Basin mine, the rib conditions with a 25° offset were still not satisfactory – an irregular rib line meant that the workforce was not confident that the rib was adequately supported, collaring the holes was difficult, and there was a large degree of hole collapse. A survey of other roadways indicated that the 35° offset was necessary to optimise the rib. The explanation may be related to the different stress field induced around an advancing coal mine roadway compared to that induced with an overburden blast in a surface mine. An analysis of the induced shearing along coal joints suggests that this could be the cause of the additional deterioration.

Secondly, the optimum layout of a surface mine avoids “noses” in the pit wall. Underground, every intersection defines either two or four equivalent noses. The implication is that even the ideal layout underground will require particular attention to the stability of the ribs near intersections.

Thirdly, the coal rib is mechanically excavated to a maximum height of about 3.5 m high and the workforce is located within 1 m of its immediate toe. In surface mines, the rock batter is blasted and then scaled, is often 20 m or more high at angles less than about 75°, and the workforce is protected in cabins and there is an imposition of no-go zones. All this means that there needs to be a different appreciation of the specific hazards. Toppling and buckling can produce relatively small blocks falling from high (potential head, neck and chest exposure), while slides and wedges provide large volumes of rock directed at the lower torso and legs.

**Stress controlled**

It is now believed that the stress field in coals is significantly different from that in the stone, with the major principal stress being approximately vertical with the horizontal stresses related to the Poisson’s Ratio effect (Seedsmann 2004). As a coal seam is depressurised by the drainage of water ahead of mining, the coal compresses in response to the increase in effective stress. As it compresses it decouples from the overlying stone and any “tectonic stress” is redirected into the stone. As the area of coal compression extends outwards, the overlying stone sags and reloads the coal. Horizontal stresses are induced under this lithostatic loading condition, with their magnitude related to the Poisson’s Ratio of the coal. As a result, the horizontal stresses in coal can be as low as 20% of the vertical stress. The vertical stresses may also increase with time depending on the rate of drainage and mining advance – there is evidence that at the face in a virgin coal seam the vertical stress may be about 50%
of that related to simple overburden loading and that it increases to the expected level with a few hundred metres outbye of the face. Possible depressurisation of the mined seam by adjacent workings in the same seam or in seams above and below needs to be considered. Retreat of the longwall will increase the vertical stresses near the face.

This alternative stress model for coal, when combined with the evolving understanding on the behaviour of brittle rock, provides a better understanding of mining-induced fracturing (MIF) that was the focus of ACIRL research in the 1980s (O’Bierne et al., 1986). Coal can be considered to be a brittle material and hence the ideas from Canada (Martin et al., 1999) on brittle rock can be applied – the key one being that cohesion and friction are not mobilised simultaneously at low confining stresses. The implication is that the failure criterion for coal near to an excavation should be based on the Hoek Brown criterion with m = 0 and s = 0.11.

The combination of the stress and brittle coal models leads to the prediction that the onset of poor ribs occurs when the unconfined compressive strength/vertical stress ratio exceeds 0.27. The strength of coal in this case can be the laboratory values, and the range of strength is 10 MPa for high quality coking coal to in excess of 30 MPa for some of the dull thermal coals. Note that the onset of rib deterioration may be progressive if the vertical stress magnitude increases outbye as proposed above. The predicted rib failure is localised initially at the roof and floor corners, with the tendency to define a vertical failure surface inside the rib (Figure 5). The similarities to Figures 2 and 3a are striking.

MIF can interact with existing joints and define slender columns of coal that may fall, or it may form extensive slabs (Figure 3a). It is possible that the common occurrence of “buckling” reported by Colwell (2005), which according to the definitions in this paper would require an alignment of roadway sub-parallel to the coal joints, is actually MIF. The deterioration of the rib at the roof line can be problematic as it is an area where it is difficult to install support. At UCS/cervical stress ratio close to 0.27, it may be possible to use spot bolts but as the ratio increases the coal may need skin restraint to prevent collapse between the bolts.

**Bolt designs**

With these collapse modes, the approach to support design is to locate the anchorage behind the identified failure surface. Anchorage lengths can be designed using civil engineering ground anchorage approaches (Littlejohn and Bruce, 1975), with high factors of safety greater than 2.5 to allow for the uncertainties regarding resin loss (Figure 6).
It is important to note that many of the failures are not deep-seated and are in fact very shallow. If the bolts are not fully encapsulated right to the collar, there needs to be attention paid to the durability of the bolt/plate assembly – especially for cuttable bolts.

Limitations of the drill rig locations may mean that bolts cannot be located in the ideal places. Rigid straps and panels may assist.

**AN EXAMPLE**

This example is a composite of a number of recent mining operations in both New South Wales and Queensland.

Consider a development roadway in a steaming coal mine with a coal UCS of 20 MPa and roadways at depths of 150 m increasing to 250 m. There has been mining in the overlying seam 30 m above. The coal seam dips to the right so there is a bias in the relative dip of the coal joints to the left. The extreme dip of the joints is $65^\circ$. The joints are generally widely spaced but there are zones where the joint spacing is about 0.3 m. The roadway is aligned at $20^\circ$ to the strike of the joints. There are two mid-seam clay bands.

The coal seam is 5 m thick and the development roadways are 3.4 m high. The bolter miner has rib bolters that can install bolts above 1.7 m from the floor and within 0.36 m of the roof (Figure 7).
At depths less than 200 m, the ribs should behave in a blocky mode. On the left hand rib, there is no buckling hazard because the joints are adequately widely spaced. The left hand ribs can be adequately supported using spot bolts located above the clay bands. Given the use of a bolter miner, it must be assumed that the wedge defined by the flattest dipping joint on the right-hand side will not fail near the face and must be supported with the rib bolts. The top of the right hand rib can be adequately supported with spot bolts but there is a problem with how to retain the lower rib below the clay bands. Longer bolts are needed for the right hand rib than for the left hand rib. Because of the alignment with respect to the cleat, some difficulties in installing rib bolts are to be expected (hole collapse, loss of resin). For the steeper dipping joints, there is a need to be sure that the bolts are fully encapsulated right to the collar or there is an effective plate – this is a particular concern if cuttable bolts are being used. Outbye deterioration of the rib is not anticipated for this mine because previous mining has already altered the stress field in the coal. The retreat of the longwall may induce a change in conditions related to the increase in vertical stresses in the abutment zone at the maingate corner. The impact will be similar to the rib conditions on driveage beyond 200 m depth.

Beyond 200 m depth, the ribs will start to deteriorate at the face with the onset of MIF. This will make the collaring of the bolts more complicated for the face crews. The spalling at the roof and floor corners will change the hazards significantly. The MIF at the floor will undercut the coal up to the clay bands and exacerbate the problem introduced by the lower bolt location. The MIF at the top of the ribline may need to be restrained somehow. At these depths the introduction of straps and panels may assist in controlling the coal between the bolts. Spray membranes will assist in controlling the rib until the pillar is rib is compressed by the next vertical stress change.

SUMMARY

Ribs present a range of hazards related to the joint structure of the coal and the onset of brittle failure. The failure modes can be anticipated and there are options available in terms of mine layouts and the specification of selection of the bolters and bolter miners. Rib support designs can be checked against a range of simple failure mechanisms. It is recommended that the ideas in the paper be used as a complement to the empirical methods, and that the choice of input values be recalibrated to the coal strength and joint orientations and joint spacing at each site.
ACKNOWLEDGEMENTS

The ideas presented in this paper have evolved during consulting projects conducted by the author in his position with Seedsman Geotechnics at a range of mining sites in New South Wales and Queensland. The desire of the various mine managers and their workforces to see an improvement in rib conditions provided the impetus for the work. Their encouragement is acknowledged. This paper is part of an ACARP project C14029 being managed by the University of Wollongong – the project is developing and documenting analytical tools for the specification of roof and rib support.

REFERENCES

RECOVERING FROM MAJOR ROOF CAVITIES ON THE LONGWALL FACE – A CURRENT PERSPECTIVE

Russell Frith

ABSTRACT: Recovering from roof cavities on the longwall face is an endemic aspect of longwall mining and this is especially the case today as longwall faces are wider and higher than they have ever been, yet powered support ratings are effectively technology constrained. There is little doubt that the ever-increasing dimensions of longwall faces are consequently increasing the likelihood of major roof falls occurring, especially in those more challenging geotechnical environments containing poor immediate roof conditions at depths of cover greater than 250 m. As a result, the efficiency and safety of longwall roof cavity recoveries is being given increased attention and is more relevant than ever to the success of the coal industry. This has led to the now almost universal use of cavity fills during the recovery of major roof falls on the longwall face, albeit that there is industry discussion regarding what constitutes ideal properties for cavity fill material. The paper discusses the geotechnical reasons why it is believed that major roof falls on longwall faces are becoming more likely with time and defines a conceptual geotechnical model for the cavity recovery process and the inherent ground control problems involved. Furthermore the paper considers the needs of mine operators during the cavity recovery process and how these can be best achieved. Specifically the paper contrasts foaming cements and phenolic foams as the two main generic types of cavity fill and ranks them according to such parameters as strength, foaming properties, rate of cavity fill, material cost, effectiveness, safety and overall cost effectiveness. The paper concludes that on a holistic basis, phenolic foams are the more suitable means of cavity filling on the longwall face, accepting that foaming cements are also an effective medium. However in the final analysis neither type of cavity fill is cost effective when compared to the benefits of preventing such cavities occurring in the first place through layout design, equipment maintenance and good operating practices and this is the major point of the paper.

INTRODUCTION

A primary focus of strata control practice at every longwall mine should be geared towards preventing major cavities on the longwall face.

The following controls are all of relevance and require due consideration in the longwall mining assessment and geotechnical design process:

- panel layout,
- panel width,
- extraction height and working horizons,
- depth of cover
- chain pillar design,
- overburden lithology and weighting,
- immediate roof competence,
- major geological structures,
- powered support rating and design
- powered support maintenance, and
- operational face management (including when to stop and apply pro-active remedial measures)

Frith (2005) provides a detailed commentary on many of the geotechnical aspects involved with instability on the longwall face, with the following summary points being given herein for reference purposes:

(i) certain geotechnical environments are more conducive to effective longwall extraction than others and some are totally unsuited,
(ii) longwall faces are generally getting wider and extraction thicknesses and depths of cover increasing,
(iii) powered support ratings are technology constrained.

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(iv) roof fall potential on the longwall face is strongly linked to face height and either face width or depth of cover (depending on the width to depth ratio of the panel), as well as the competency of the immediate roof measures.

(v) production requirements are ever-increasing and unit cost requirements are ever-decreasing.

Therefore, the coal industry is possibly at a point whereby major roof falls on the longwall face are becoming less tolerable, but conversely more likely in general terms. That is not to say that there will soon be “outbreaks” of major roof falls across the industry, but the geotechnical factors are being driven in a direction that in many instances will incrementally reduce roof stability on the face.

In the past twenty years our knowledge as to why major roof falls occur on the longwall face and more importantly how to go about preventing them, has improved significantly. This has been largely due to experience based improvements in powered support design, operational face management practices and knowledge regarding the role of the geotechnical environment. Nonetheless almost all longwall mines will from time to time be faced with a major cavity on the longwall face, such that its safe and efficient recovery becomes the key short-term focus of the mine.

Anyone who has been involved with longwall mining for more than 20 years will undoubtedly have war stories to tell regarding setting timber cribs in roof cavities above longwall powered supports. In fact, in the Longwall Larrikins section of the International Longwall News website, in response to the question “what was your scariest time in a coal mine?” Nick Fowler is quoted as saying “on top of shields, timbering cavities. I hated it, we were mad and the use of foams, grouts and resins couldn’t come soon enough”.

Current OH&S considerations would never allow such a practice to continue (even if someone mad enough was available to do the work!). However there is on-going industry discussion in regards to the safest and/or most effective/efficient method(s) by which major roof cavities can be recovered, this being largely based on two distinctly different commercial products, namely foaming cement (e.g. Tekfoam) and phenolic foam (e.g. Rocsil foam).

To provide credible comments on this subject, an appreciation of the geomechanics of longwall faces and the problems faced by operators once a major cavity has formed is required and this is a major focus of this paper.

**ELEMENTS OF A MAJOR ROOF FALL CAVITY**

Irrespective of the primary causes (eg periodic weighting, altered strata around a major fault etc.) of any particular major roof fall on a longwall face, the resultant face conditions (as seen in Figure 1) contain a number of common features. Figure 2 illustrates these features in schematic form for ease of illustration, which are as follows:

1. The coal face hading away from the AFC and powered supports, thus increasing the effective tip to face distance.
2. Highly fractured coal for some distance ahead of the face.
3. A large amount of broken material (coal and rock) with an angle of repose sat on the AFC. The AFC can be stalled under this load and/or contain large rock lumps that require blasting to allow conveying from the face.
4. Fractured and broken immediate roof measures just ahead of the coal face.
5. Large lumps of rock (that would require blasting if they fell onto the AFC) balanced precariously on top of the powered supports.
6. Poorly aligned powered support canopies.
7. The cavity itself with other potentially unstable pieces of material around its perimeter.

Assuming that the shearer is not buried under fall material and has been removed from the immediate fall area, it is either all or the majority of these features that the mine operator faces when attempting to recover the face from beneath a cavity and so re-establish normal face production.
Fig. 1 - Typical face conditions during a major roof fall

Fig. 2 - Schematic illustration of the primary features of a major roof fall on the longwall face
COMMON PROBLEMS EXPERIENCED DURING RECOVERY PRIOR TO THE USE OF CAVITY FILL

The scenario illustrated in Figure 2 was that commonly faced by longwall operators prior to the advent and use of monolithic roof cavity fill materials. As mentioned in the introduction, a traditional approach was to build timber cribs above the powered support canopies and this was a common practice for many years. However when this was discontinued due to safety concerns, face recoveries had to be undertaken without a cavity fill of any form in place and this led to a number of common problems as will now be described.

(a) AFC operational

The first requirement of the face recovery process is to ensure that the AFC can be run as and when required, as without it there is no ability to move the face forward. After a large fall affecting a significant length of the longwall face, it is common that the AFC is either overloaded and stalled (thus requiring the hand removal of large quantities of material from the AFC) or as a minimum, the breakage (often via blasting) of large rock lumps into smaller pieces is necessary.

Large lumps balancing on top of the powered supports commonly have to be dropped onto the AFC (to allow the powered support canopy to be re-aligned later on) and blasted to allow removal by the AFC.

Returning the AFC to an operational status in itself can be a time-consuming and very frustrating process, this reiterating the point that “prevention is undoubtedly better than cure” when it comes to this particular strata control problem.

It is noted that prior to the stabilisation of both the coal face and roof strata ahead of the face, running the AFC for any period of time was ill-advised as it inevitably resulted in more broken material falling down onto the AFC, potentially initiating more hand work or blasting of large rock lumps. The requirement at this stage in the process is for the AFC to be able to run, not to necessarily run it for any significant period of time.

(b) Stabilisation of fractured coal and/or roof strata ahead of the face

As just mentioned, running the AFC with an unstable coal face and immediate roof ahead of the face was usually counter-productive without cavity fill in place, as it commonly resulted in more broken material falling out on the AFC.

Methods of attempting to stabilise the coal face and immediate roof are either the injection of some form of consolidation agent (eg polyurethane or cement grout) and/or the installation of ground support elements.

By far the most common method of consolidation used is polyurethane (PUR) injection largely due to its expansive and adhesive (i.e. its sticks to rock and coal) properties (Figure 1 shows PUR injection lances coming out from the broken coal face).

Injected cement grouts are generally non-expansive and have little or no adhesive properties, so that they act only as fillers. In a situation whereby there is a large free face (i.e. the walls of the cavity itself), improved stability close to the free face is unlikely to be achieved by just void filling. Adhesive properties are required, hence the more common use of PUR in such situations.

However PUR does have significant downsides which can detract from its effectiveness in this situation, the major technical downside being almost entirely due to pumpable volume limits per hole as a result of the exothermic reaction and heat generated during its curing process. Due to legitimate concerns regarding the heat generated by PUR and its ability to potentially promote combustion of broken coal, regulatory authorities in Australia have imposed volume limits on the amount of PUR that can be pumped into broken coal at any one time. This is typically 200 litres per hole, although it is understood that the Queensland Mines Department has more recently removed this restriction in those situations whereby the PUR will not be injected into coal.

When one considers the likely volume of open voids within both the coal seam and roof measures ahead of the longwall face containing a major roof cavity, 200 litres per hole is a very small quantity indeed. Experience would suggest that in many cases, the void space present is anything but filled (as evidenced by a common lack of back pressure on the pump injecting the PUR), so that a significant portion of broken material remains isolated from a competent rock mass at the cessation of PUR injection activities.
Additional PUR injection campaigns can be undertaken to further fill void space, but only after a defined period of time to allow heat generated by the first injection campaign to be dissipated. This is often counter-productive as face conditions demonstrably deteriorate with time (see Frith and Stewart 1994) so that more strata fractures and resultant void space are being created during the waiting period.

Overall, PUR injection is anything but an ideal method of attempting to re-consolidate broken strata ahead of a longwall face, but at the current time it is the best method available. Whilst its success cannot be guaranteed, it is almost certain that attempting to recover a major cavity without it will substantially increase the likelihood of the cavity propagating further and so increase the delay to normal production being resumed.

In terms of installing ground support elements, by far the most common method is to leave the PUR hollow injection rods in the hole, these acting as a crude type of spile. These can be either steel or made of cuttable material, although caution is recommended in the use of steel injection rods as they will cause significant difficulties if they should end up on the AFC.

(c) Running the AFC
With the AFC able to be run and the coal and roof ahead of the face consolidated (within the limits of what can be achieved), the next stage is to run the AFC as part of clearing away the broken material and so attempting to move the face forward.

If the consolidation process has been fully effective, the AFC will remove the broken material from the face and no more lumps of coal or stone will fall onto the face. However this was rarely if ever the case prior to the use of cavity fill as strata consolidation was almost never sufficiently effective to allow this to happen. As a general rule, running the AFC to clear broken material from the face will simply cause more broken material to drop down, thus requiring that the process of lump breakage and removal recommence.

It is typically a very frustrating process of removing broken material and re-clearing the AFC before the face can be advanced in an attempt to cut under the broken roof ahead of the face and so form a stable lip that the tips of the powered supports can be set against.

(d) Establishing a competent lip
With the AFC running and clear of large lumps of broken rock, the face can be moved forward with the intent of having the shearer cut under the re-consolidated roof in an attempt to establish a stable lip that the powered support canopy tip can be set against.

This is rarely done for the entire cavity length in one go, the preferred method being to commence at either or both ends of the cavity and establish a lip adjacent to an already stable immediate roof. In this manner a fully stable immediate roof can be re-established over a small number of, rather than in a single shear.

As with the initial re-running of the AFC, the success or failure of attempting to re-establish a lip beneath consolidated roof will depend on:

(i) whether the immediate roof was in fact fractured and broken in the first place – NB just about all major falls have a finite outbye limit and a competent roof will eventually naturally form, regardless of what remedial measures have been applied, and
(ii) the effectiveness of the consolidation measures injected into the immediate roof material.

As already stated, due to the volume limits associated with the pumping of PUR in close proximity to broken coal, it is hit and miss as to whether all of the pre-existing void spaces in the immediate roof have been effectively filled and adjacent rock fragments “glued” together. With significant void space remaining and the presence of a significant free face (i.e. the cavity walls), re-establishing a solid lip that the tip of the powered support can be set against can be a difficult process, failure to do so often exacerbating the roof cavity and allowing further broken material to fall down onto the AFC.

Hence the process of fall recovery has to start again, frustrating mine operators and incurring further costs associated with both the application of remedial measures and lost production.
(e) Summary

If the described process and problems associated with recovering from a major roof cavity without void filler in place are reviewed on an engineering basis, the following conclusions can be readily drawn:

(i) PUR injection and support elements (i.e. spiles) in isolation cannot (and in fact should not) be relied upon to internally “glue” broken roof material together to form a re-consolidated roof mass that is readily suitable for re-establishing a competent lip on the face.

(ii) The key problem is that the roof cavity itself forms a very effective “free face” within the immediate roof horizon (i.e. effectively creating a rock cantilever ahead of the powered supports) that prevents any form of confining or stabilising horizontal compressive stress being generated. Therefore roof stability has to rely on the tensile strength of the immediate roof measures, which will be minimal if they contain open voids, as well as natural vertical joints etc.

(iii) Elapsed time undoubtedly works against longwall face conditions, so that the need to undertake several iterations of the recovery process by definition, makes an effective recovery harder to achieve.

(iv) Some of the required tasks in a major fall recovery are by necessity undertaken in close proximity to large amounts of rock material that is essentially, uncontrolled. Whilst the history of major safety breaches during such recoveries is quite reasonable (e.g. fatalities are an extremely rare event as are even major injuries), the fact remains that the recovery process exposes mine workers to higher risk levels than would be encountered in most other day to day mine operations. The reasonable safety record is possibly as much to do with the low time-based exposures (i.e. major fall recoveries do not occupy a large proportion of the available time on a longwall face) rather than the fact that it is a relatively safe operation to undertake.

When all of the relevant points are analysed, the conclusion can be quickly reached that effective roof fall recoveries should benefit significantly from the use of cavity filling. In the past this was achieved via the construction of timber cribs above the powered supports, which over time has been replaced by the application of pumpable monolithic type fillers.

The subject of cavity filling is therefore the next logical topic of discussion.

THE USE AND APPLICATION OF CAVITY FILLING

Figure 3 provides a schematic representation of the potential impact of a cavity fill as part of a roof fall recovery, it being a development of the earlier used Figure 2. Figure 4 and 5 contain photographs of the longwall face during a cavity fill application using Rocsil foam (courtesy of Wilson Mining).

Fig. 3 - Schematic illustration of the impact of cavity fill in and around a major longwall roof fall
Fig. 4 - Rocsil foam having been applied to a roof cavity on the longwall face

Fig. 5 - A full face of Rocsil foam following a major cavity fill application
The main points that emanate from the use of a cavity fill are as follows:

- Fractured material in the roof measures ahead of the face is to some degree confined by the cavity fill material (especially foaming or expansive fillers), thus offering it some additional stability. Therefore when the AFC is run and broken material is removed from the face, the potential for more material to fall out onto the AFC is greatly reduced.
- Similarly, the fill material offers horizontal confinement to any broken reposed material that may want to flow onto the AFC as well as the coal face itself, providing additional stability to both.
- Any large rock lumps sat on top of the powered supports will to some degree by stabilised by the fill. Such lumps will also tend to be pushed backwards towards the goaf as the powered supports advance, rather than riding forward with the powered support. This process should eventually cause them to fall into the goaf behind the face rather than onto the AFC during recovery operations.
- The powered supports will have a roof horizon to set against before the tips reach the lip of the roof cavity. Even though they may not be able to be set at full set pressure (due to the inherent strength of the fill material – see later), there are advantages in having a flat, albeit false roof to work to when advancing the face forwards towards the roof lip.

When the ability of cavity filling to mitigate against the primary risks associated with the recovery of a major roof fall on a longwall face is considered, the potential benefits are self-evident and they explain why there has been such a rapid take-up of monolithic pumpable cavity fillers.

The question therefore remains as to which (if either) of the generic cavity filler types (foaming cements or phenolic foams) is best suited (in general terms) to effective roof fall recovery.

**GENERAL COMPARISON OF FOAMING CEMENTS AND PHENOLIC FOAMS**

In order to undertake a comprehensive comparison of these two generic cavity fill technologies, their varying attributes in the following areas need due consideration:

- Short-term or instantaneous strength
- Levels of foaming - impacting directly upon the quantity of material needing to be transported to site to fill a cavity of a given volume and also well as speed of cavity filling
- Shuttering requirements on the face
- Flow properties (in particular thixotropic properties that may limit the gravity driven flow of material into unwanted areas – eg AFC sigma section etc.)
- Environmental issues (eg noxious or poisonous fumes being emitted into the mine ventilation)
- Rates of bulk application
- Pumping distances

Each of these particular parameters will in some way impact directly or indirectly upon the three issues that will be of most interest to the mine operator:

1. Safety of operations
2. Effectiveness in terms of recovering the face
3. Total cost to the operation (including minimising production losses).

It is in this context that the varying attributes of foaming cements and phenolic foams will be compared, a summary being given in Table 1.

Based on the contents of Table 1 the following observations are made:

- Higher rates of fill emplacement and reduced bulk material usage being claimed by the suppliers can be logically linked to the higher foaming properties of phenolic foam.
- The quicker emplacement of phenolic foam cavity fill should have a positive impact in the context that face conditions typically deteriorate with time, especially in the early stages of a cavity developing. Therefore being able to attempt to re-establish a competent lip faster has geotechnical advantages.
- The faster curing time and thixotropic nature of phenolic foam are the logical reasons as to why shuttering requirements are significantly less as compared to foaming cements. This not only increases
• the speed by which shuttering can be erected but also effectively eliminates the need for mine operators to work on the AFC when erecting it.
• There is little difference between the two in terms of environmental issues. Both have safety issues to manage in terms of both handling and emplacing the material.
• With longwall faces becoming incrementally wider, the significantly increased pumping distance of phenolic foam should become an ever important advantages favouring the use of phenolic foam.

Table 1 - Parameter comparison between foaming cements and phenolic foams
(based on publicly available information from product suppliers)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Foaming Cements (eg Durafoam, Tekfoam)</th>
<th>Phenolic Foams (eg Rocsil Foam)</th>
</tr>
</thead>
<tbody>
<tr>
<td>short-term strength</td>
<td>0.15 MPa (2 hours)</td>
<td>0.1 to 0.2 MPa at 10% deflection in 5 minutes at 15°C and 2 minutes at 25°C</td>
</tr>
<tr>
<td></td>
<td>0.25 MPa (1 day)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.40 MPa (7 days)</td>
<td></td>
</tr>
<tr>
<td>foaming characteristics</td>
<td>12 to 14 times original volume</td>
<td>30 to 35 times original volume 40 kg/m$^3$ of cavity filled positive pressures generated against cavity walls</td>
</tr>
<tr>
<td></td>
<td>100 to 140 kg/m$^3$ of cavity filled</td>
<td></td>
</tr>
<tr>
<td></td>
<td>nothing stated regarding positive pressures being generated on cavity walls</td>
<td></td>
</tr>
<tr>
<td>bulk rate of application</td>
<td>up to 24 m$^3$/hour (Poland)</td>
<td>50 to 75 m$^3$/hour up to 300 m$^3$/shift</td>
</tr>
<tr>
<td></td>
<td>up to 76.5 m$^3$/shift (USA)</td>
<td></td>
</tr>
<tr>
<td>flow properties</td>
<td>nothing stated in regards to flow properties</td>
<td>thixotropic to minimise potential for AFC blockages due to flow of material under gravity</td>
</tr>
<tr>
<td>shuttering requirements</td>
<td>structural type frameworks to contain emplaced material during the curing process</td>
<td>brattice and pogo rods installed from behind the AFC spill plates</td>
</tr>
<tr>
<td>environmental</td>
<td>non-toxic and non-flammable, precautions required for various forms of person contact</td>
<td>FRAS, relatively benign when solid, precautions required whilst handling constituents and applying at face</td>
</tr>
<tr>
<td>pumping distances</td>
<td>up to 200 m</td>
<td>up to 600 m</td>
</tr>
<tr>
<td>other</td>
<td></td>
<td>exothermal reaction up to 187°F which is of no significant concern in coal mining</td>
</tr>
</tbody>
</table>

Cleary then in a number of technical areas, phenolic foam has distinct advantages as compared to foaming cements when used as a cavity filler on a longwall face. The one technical area where it may possibly be at a disadvantage is in the area of short-term strength and this is worth considering further before reaching any firm conclusions.

SIGNIFICANCE OF LOW SHORT-TERM STRENGTH OF CAVITY FILL MATERIAL

A modern-day longwall powered support will typically generate a canopy support load density of around 100 tonnes/m$^2$ or 1 MPa as a pressure. This is an average pressure across the canopy and depending upon the configuration of the powered support, peak canopy pressures can be at least twice this at 2 MPa and greater. Therefore for the powered support to generate its maximum load capacity against the roof, the roof itself must be able to accommodate such pressures without undergoing significant yield.
Examining the short-term strengths for both foaming cements and phenolic foams, it is evident that neither comes close to 1 to 2 MPa, even after seven days in the case of foaming cement. It is noted that shotcrete will only reach 1 MPa in around four hours, even if significant doses of accelerator are used.

Therefore on the basis that many successful longwall recoveries have been achieved with both cavity fill types, the question has to be asked as to whether the strength of the fill material relative to the maximum support load density of the powered is of any real significance to facilitating an efficient face recovery?

By reference to well established surface bearing pressure principles, an allowable bearing stress can be three to six times the UCS of the bearing material (Pells, Mostyn and Walker, 1998). This is due to the confined nature of a bearing surface and the fact that true uniaxial failure conditions cannot be generated within it. Therefore as an initial comment, the UCS of the cavity fill material does not necessarily need to be equivalent to the bearing stresses generated by the powered support canopy for the full load capacity of the powered support to be developed. Certainly load densities several times higher than the cavity fill UCS can potentially be generated into the roof when the powered support is set against cavity fill.

From a face roof control point of view, the role of the powered support during normal face operations is to actively reinforce the immediate roof measures, not necessarily those above it but certainly ahead of it. It does this by limiting roof convergence above the face (hence also reducing face spall) and limiting the effective tip to face distance by having the load centre of the canopy as close to the face as possible.

It is during normal face production that the full rating of the powered support is required; in particular it’s setting load density and on-going hydraulic integrity so as to maintain leg pressures at or above set. However once a major roof cavity has formed (as shown in Figure 2) and the face has been stood for a period (so that a significant portion of the roof convergence that will occur has now done so), the need for active reinforcement of the immediate roof ahead of the face is significantly reduced. The cavity itself (even when filled) also limits the powered support generating active reinforcement action in the strata ahead of the face.

Based on the previous comments it makes good sense that the low strength of foaming cavity fill in comparison to the load capacity of the powered support, does not significantly detract from the efficient recovery of the face. During recovery operations the immediate roof ahead of the face is typically injected with PUR to try to re-consolidate it and the fill material offers some confinement to the walls of the cavity to minimise loose pieces falling out on the face. These are the main short-term controls against further roof instability once the face starts cutting again.

During the fall recovery process the powered support often only acts as a means of holding back goaf material from the working areas and pushing over the AFC to allow the face to be advanced when attempting to re-establish a lip. Neither of these functions relate to the powered support being able to exert its full load capacity against the roof, goaf material behind the supports allowing the AFC to be pushed over with the canopy off the roof in fact.

When the differences in short-term strengths between foaming cements and phenolic foams (in relation to the full load capacity of the powered supports) are considered, along with the role that the powered support plays in the recovery process, it is concluded that there is little to choose between the two cavity fill types. Other properties are of far greater significance as has been discussed previously.

**ECONOMIC EVALUATION**

One comment that is often made by mine operators is that phenolic foams are “very expensive in comparison to foaming cements”. Therefore it is worth undertaking a basic economic evaluation of a major longwall roof fall and its recovery to put this aspect into a more realistic context.

In a high production longwall operation, time is by far the most significant factor when it comes to overall economics. Production losses of $1-$2 million per day (i.e. 20,000 tonnes x $50-$100/tonne) are commonly quoted if one days longwall production is lost, but such numbers are possibly misleading when considering the financial impact of unplanned downtime.
Firstly the coal not mined during the production delay will be mined eventually, such that the loss is NPV based. There will also be other costs that are not incurred as a result of the longwall not producing (eg washing, rail freight, royalties), but these are included in the assumed $50-$100/tonne sale price.

It might be assumed for example, that the longwall face being stopped for one day will typically result in fixed costs of say $200,000 (wages, power, depreciation etc) being incurred that can never be recovered. This is assessed to be a more meaningful method of examining the financial impact of one days lost production at a high production longwall mine than simply examining revenue losses in their totality.

All of the available technical information demonstrates that phenolic foams can be applied to cavity filling at a faster rate than foaming cements. This is due to both the less onerous shuttering requirements and most significantly, the higher foaming rate so that a larger void is filled for each unit of raw product and unit time.

Based on recently obtained raw product unit cost estimates for both phenolic foam (i.e. Rocsil) and foaming cement, combined with quoted foaming ratios, it is evident that phenolic foams (at around $1000/m$^3$ of filled cavity) are approximately 2.5 times the cost of foaming cements (at around $400/m^3$ of filled cavity). Hence the common statement that phenolic foams are very expensive.

For a large roof cavity of say 750 m$^3$ it is estimated that a phenolic foam system could effect a complete fill (including set-up, emplacing shuttering etc.) in around four days, whereas foaming cements would take at least two if not three times this length of time. This has both geotechnical (i.e. face conditions will tend to deteriorate with time, especially in the early stages of a major fall) and also cost implications, the latter of which will be considered in more detail.

If the mine site costs associated with longwall face downtime are also considered (as detailed above), it is evident that the total incurred cost for the phenolic foam cavity fill at around $1.55 million (750 m$^3$ x $1000 + 4 x $200,000) is in fact significantly lower than that for foaming cement at $1.9 - $2.7 million (750 m$^3$ x $400 + 8-12 x $200,000).

Therefore in order to realistically evaluate the relative costs of foaming cements and phenolic foams, it is necessary to factor in the difference in mine site losses that are incurred due to the varying rates at which the two products can be applied.

Clearly there will be other methods by which the financial impact of a major roof fall can evaluated (eg the difference between the profit with the longwall operating as compared to the loss incurred with it stopped). The important point to make is that “value for money” should be the determining factor, not simply material cost.

**CONCLUSIONS**

The recovery of large roof cavities on the longwall face demonstrably benefits from the provision of some form of cavity fill prior to attempting to move the face forward and re-establishing a stable lip along the face. Historically this was achieved via the erection of timber cribs above the powered support canopies and this has evolved to the remote application of monolithic cavity fill materials.

The cavity fill material primarily acts to confine the perimeter of the cavity and so attempts to hold loose material in place that would otherwise fall out onto the AFC and so impede the recovery process. It does so largely through foaming action that allows the foam to fully fill the void and potentially offer an active confining pressure to any surrounding marginally stable rock.

In contrast the strength of the fill material is a secondary issue as utilising the full supporting capacity of the powered supports is significantly hindered by the presence of the cavity void in the first place and offset by the almost universal use of re-consolidation measures (eg PUR injection) in broken strata ahead of the face. Nonetheless it should not be assumed that the powered support is limited to load densities equivalent to the UCS of the cavity fill material, as the bearing type nature of the problem dictates that a bearing material (i.e. the cavity fill in this case) can accommodate bearing pressures several times its own UCS.

In comparing the relative merits of foaming cements and phenolic foams, as a general rule it is apparent that phenolic foams such as Rocsil are more suited to rapid, safe and effective roof cavity recoveries, this being largely due to their higher foaming ratios and reduced shuttering that is required to contain the fill material. This is not to
say that foaming cements cannot be used to recover such cavities (as indeed they can), simply that the properties of phenolic foams are more suited to the task.

In terms of engineering the most effective cavity recovery possible, it is important to evaluate both the cost of cavity filling and technical attributes together, in particular ensuring that the true cost to the operation includes an allowance for the different time periods taken to effect the placement of the cavity fill. Based on some broad assumptions made herein, it appears that despite being a more expensive raw material, the overall economics of phenolic foam are generally better than foaming cements.

Overall there is no guarantee that the recovery of a major roof cavity on the longwall face will be successful at the first attempt, largely as the control of a failed and broken rock mass contains a much greater level of behavioural uncertainty than an intact mass. Cavity fills are one of a series of controls that aims to improve the odds in favour of the mine operator and industry experience indicates this to be a largely effective strategy.

In the final analysis the most economic strategy of minimising the economic impact of major roof cavities on the longwall face is to maximise the effort to prevent them in the first place. Many of the controls, both geotechnical and operational, are well understood and including them within mine planning and operational management should be a key focus if the true cost of remedial measures is to be minimised to the lowest practical level.

**REFERENCES**


Frith, R C, (2005). Half a career trying to understand why the roof along the longwall face falls in from time to time? Proc. 24th Int. Conf. on Ground Control in Mining, Morgantown, West Virginia, pp 33-43.

AN INTEGRATED REAL-TIME ROOF MONITORING SYSTEM FOR UNDERGROUND COAL MINES

Baotang Shen¹, Hua Guo¹, Andrew King¹ & Murray Wood²

ABSTRACT: CSIRO has been conducting a five-year research project under the sponsorship of JCOAL and Ulan Mine to develop a real-time roof monitoring and roof fall warning system for underground coal mines. A preliminary system has been developed and successfully trialled twice in the gateroads at Ulan Mine during 2004 and 2005. The system integrates the displacement monitoring, stress monitoring and seismic monitoring in one package. It includes:

- GEL multi-anchor extensometers
- Vibrating wire uniaxial stress meters
- ESG seismic monitoring system with microseismic sensors and high frequency AE sensors.

The monitoring system has been automated and the data are automatically collected by a central computer located in an underground non-hazardous area. The data are then transferred to surface via an optical fibre cable. The real-time data can be accessed at any location with internet connections.

The trials of the system in two tailgates at Ulan Mine have demonstrated that the system is effective for monitoring the behaviour and stability of roadways during longwall mining. The continuous roof displacement/stress data have showed clear precursors of roof falls. The seismic data (event count and locations) have provided insights into the roof failure process during roof fall.

The sensor type and real-time communication system are flexible and can be tailored to meet site-specific monitoring needs.

INTRODUCTION

Roof fall is a major hazard in underground coal mines in Australia. It can cause fatalities, injuries and significant economic losses. Fatalities and injuries due to roof falls in Australian underground coal mines have been significantly reduced in recent years, thanks to the efforts of mine operators and industry regulators. Production loss due to roof falls continues to be a major industry concern for underground coal mines. Roof falls have caused the stoppage of longwall mining and/or roadway development for days or weeks.

Real-time monitoring and early detection of imminent roof fall allowing preventative action to be taken, will increase safety margins and bring significant economic benefit to the mining industry.

A research project was established in 2002 under the sponsorship of JCOAL and CSIRO to develop a roof fall monitoring and warning system for underground coal mines. The project is of 5 years duration total. During the past 4 years, an integrated real-time roof monitoring system has been developed. The system has been successfully trialed twice at the tailgate of longwall panels 20B and 21 at Ulan Mine. This paper presents the key feature of the system and the results from the first field trial at LW20B. More details can be found in Guo et al. (2004).

INTEGRATED REAL-TIME MONITORING SYSTEM

The integrated CSIRO roof monitoring system consists of:
(1) Extensometers (measuring roof displacement);
(2) Stress meters (measuring stress change in roof and rib);
(3) Seismic sensors (detecting seismic events and mapping roof damage);
(4) Real-time data acquisition and communication system.

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Extensometer

GEL extensometers manufactured by GEL Instrumentations Pty Ltd have been used (see Figure 1). The extensometers used have 6 anchors which were specifically made for CSIRO. The technical specifications of the 6 anchor GEL extensometers are given below:

- Measurement range: 0 – 170 mm
- Accuracy: 0.5 mm
- Borehole diameters: 27 mm – 55 mm

Stress sensors

The stress monitoring device used in the monitoring system is the 4300 EX vibrating wire stress meter (Figure 2) supplied by Geokon Geotechnical Instrumentation. It measures the stress change (not the absolute stress). The features of this type of stress meter include:

- High sensitivity (sensitivity = 0.014-0.07 MPa);
- High range (compression = 0-70 MPa, tension = 0-3 MPa);
- Simplified installation. Borehole grouting is not required;
- Corrosion resistant;
- Waterproof;
- Long-term stability;
- Suitable for remote reading and automatic logging.

The required borehole diameter is 37-39 mm.

Seismic monitoring system

The seismic system includes seismic sensors (both low and high frequency) and a data acquisition system. The new acquisition system is the ESG HMSi integrated seismic monitoring system (http://www.esg.ca/home/products/hmsi1.htm) modified to enable low-frequency microseismic (MS) and high-frequency acoustic emission (AE) data acquisition cards to be combined on one chassis. This system can run on a standard PC under the MS Windows operating system. The system is capable of acquiring eight channels of MS data, at a sampling rate of up to 40 kHz, as well as four channels of AE data, at a sampling rate up to 40MHz. Extra cards can be installed to increase the number of MS and AE channels available.

There are two different types of sensors used in the monitoring system: the low-frequency MS sensors, which includes uniaxial and triaxial accelerometers, and high-frequency AE sensors with built-in preamplifiers for the AE signal detection. The MS sensors are used to detect larger events with fracture radii of tens of centimeters or greater, whereas the AE sensors are aimed at the small, grain-to-centimetre sized fracturing events.

The MS accelerometers used are the A1030 and A3005 accelerometers from ESG with a frequency range of 50Hz to 5 kHz, and 3 Hz to 8 kHz, respectively. The R6 sensors used as ultrasonic sources, and the R6i sensors, used as AE receivers, are made by Physical Acoustics Corporation. They have an operating frequency ranging from 40 to 100 kHz.
System integration and communication

The three components (extensometer, stressmeter and seismic system) of the monitoring system were integrated as shown in Figure 3. The extensometers and stress meters installed in the gateroad (hazardous area) are cabled to a data logger via safety barriers. The data logger logs the data at a specified time interval (30 minutes during the first field trial) and passes the data to the seismic data acquisition system. Thus the seismic acquisition system obtains all the seismicity, displacement and stress data. In the trial at Ulan, these data were transferred from the acquisition system in the underground to the surface via an optical fiber cable, then to CSIRO’s office at Brisbane via the internet and Xstrata and CSIRO networks.

Fig. 3 - Integrated CSIRO roof monitoring system.

REAL-TIME MONITORING OF TAILGATES AT ULAN MINE

A trial of the integrated monitoring system was conducted at LW20B tailgate at Ulan Mine in 2004. A roadway length of 800 m from 1 C/T to 8 C/T was monitored with special focus on the 4th cut-through (4 C/T). The monitoring layout is shown in Figure 4.

Displacement sensors

A total of 12 GEL extensometers were installed in the roadway roofs, eight of which were located at the centre of the intersections at cut-throughs 1 - 8, the rest were located at the roadway centre close to 4 C/T. Figure 5(a) shows the anchor position of each extensometer.

Stress meters

A total of 21 stress meters were installed in three roof and two pillar locations. At each roof location, three stress meters were installed in different directions in the horizontal plane to ensure the measurement of the two dimensional stress change in the roof rock. Stresses at different locations into the roof were measured, see Figure 5(b). At each pillar location, three stress meters were installed at a distance of 2 m, 5 m and 10 m from the roadway rib, respectively. They were designed to measure the vertical stress change in the pillar during longwall mining.
Fig. 4 - Layout and sensor locations of the 1st field monitoring trial at Ulan Mine

Fig. 5 - Sensor locations in the roadway roof
Seismic sensors

A total of 12 seismic sensors were installed in two locations. 11 of them were installed at the fourth cut-through (4 C/T) where the roof fracturing and damage during mining was monitored in detail. One sensor was installed at the sixth cut-through (6 C/T) to monitor the initial caving during the start-up of LW20B at 8 C/T. Figure 6 shows the sensor arrangement at the fourth cut-through (4 C/T).

![Fig. 6 - Seismic sensor array geometry in roadway roof at the fourth cut-through](image)

All the sensors were installed during the period from 25 April to 1 May, 2004. Mining at LW20B commenced on the 12th of May, 2004 at 8 C/T. On the 14th of May 2004 when the longwall face progressed about 20 m, the longwall roof started to cave behind the longwall chocks. The longwall face reached the 4 C/T (where the monitoring was concentrated) on the 11th of June, 2004. The longwall panel was completed in August 2004 at 1 C/T.

Upon the completion of installation on the 1st of May 2004, the extensometer and stress meter monitoring commenced. The data logger was set to record and store the data every 30 minutes. The stress monitoring continued until the 12th of June 2004 when the last stress meters at 4 C/T - 20 m were buried. The displacement monitoring continued to August 2004 when the whole Longwall panel 20B was mined.

The seismic system commenced operating on the 7th of May 2004 after a problem with the hardware was fixed. The seismic monitoring continued until the 11th of June 2004 when the longwall face reached the monitoring location (4 C/T) and the roof collapsed. Most of the seismic sensors were damaged or destroyed during the 10th and 11th June.

During the period of monitoring, data were collected daily from CSIRO’s office at Brisbane. The data were processed, analysed and plotted daily. They were then sent back to Ulan Colliery with comments on the roof condition to help their strata control and mining operations.

Overall, the monitoring ran smoothly and was very successful. For the first time in Australia, we have achieved a remote, real-time, continuous, and integrated monitoring of roof behaviour during longwall retreat in an underground coal mine.

**MONITORING RESULTS**

The results from the first field trial of the integrated real-time monitoring are extracted and analysed in this section. The following aspects directly related to the strata behaviour during mine operations are discussed:

- Initial caving
- Caving progress
- Tailgate roof displacement and stress
• Pillar stress
• Integrated monitoring of roof behaviour

Initial caving

Longwall mining at LW20B started at 8 C/T. The roof behind the longwall face did not cave until the face advanced by about 12 m. At this time, the roof span behind the chocks is about 20 m after adding the initial width of the installation road (8 m). Figure 7 shows the response of the stress meters at 4 C/T (top figure), extensometers at 8 C/T (bottom figure) and the seismic sensors at 6 C/T and 4 C/T (in both figures) during this period.

Fig. 7 - Detailed monitoring results during longwall start-up and initial caving

The mining started on 12/05/2004 11:00am. The operators observed the initial caving starting at around 13/05/2004 0:00 am. The following records were extracted from the mine shift report:

1. 7:50 pm-1:20 am (chainage=690-688 m): Chock leg pressures are high (300-400 bars); Goaf starts to form at the middle of the panel face.
2. 1:27 am-6:22 am (chainage=688-685 m): Goaf has fallen over nearly the entire length of the face. It still hangs up near the maingate and tailgate.
3. 11 am (chainage=685-683 m): Large chock converging movement occurs from mid face toward the tailgate

The roof stresses as monitored at 4 C/T, 400 m away from the caving activity, showed a clear response to the above events of caving. At point 1 (caving started), the stress parallel to the roadway started to decrease. At point 2 (caving propagated), the stress at 45° angle showed a sudden drop. At point 3 (cave completed), the stresses in all the three monitored directions rebounded sharply.

The monitored displacement at 8 C/T showed a rapid roof movement between points 2 and 3, confirming that the caving had propagated from the mid-face to the tailgate during this period.

The seismic sensors at 6 C/T and 4 C/T received an increased number of seismic events at the end of the initial caving. The peak of the seismic event count corresponds to the sharp stress rebound. Note that the seismic system during this period was frequently interrupted by unexpected computer rebooting. Therefore, not all the seismic events were recorded.

The monitoring results demonstrated that the stress meters are adequately sensitive to pick up the caving activities 400 m away from the monitored location. The changes in the roof stress, however, were very small (<0.05 MPa). Only the automated and continuous monitoring system, as used, could pick up the subtle changes. Traditional
manual monitoring with daily or weekly reading frequency will not be able to show the subtle changes on its data chart.

Caving process

A typical variation of the monitored roof stresses with the longwall face position is given in Figure 8. The variation of the number of seismic events received against the longwall face position is shown in Figure 9.

As seen in Figure 8, there is a close correlation between the monitored roof stress and the longwall face position when the longwall face progressed from 8 C/T (chainage = 700 m) to 5 C/T (chainage = 300 m). This correlation diminished when the face was within 100 m from the sensor position at 4 C/T - 20 m (i.e. between 3 C/T and 4 C/T, or chainage of 200 m-300 m).

The seismic event counts also showed a clear response to the mining progress (Figure 9). When longwall faces progressed, increasing seismic activity was recorded. When the longwall face stopped for a prolonged period, little seismicity was recorded. The monitored seismicity peaked when the longwall face was about 20 m from 4 C/T where most of the seismic sensors were located. It is interesting to note that, when the longwall face was closer than 20 m from the 4 C/T, the monitored seismicity reduced.

The close correlations between the seismicity/stress and the position of caving may be used to study the development process of longwall caving. For instance, if the monitored results suddenly deviate away from an
established trend when the longwall face is still progressing, it may indicate that the caving of the roof is not developing normally. The caving of the immediate coal roof can be observed from the longwall face, but the behaviour of the overlying sandstone roof may not be easily observed. Together with other operational data (such as the chock pressure and convergence), the monitoring data could alert the operators if the sandstone roof hasn’t caved smoothly.

**Tailgate roof displacement and stress**

The monitored roadway roof displacements at two selected locations are plotted against their relative distance to the longwall face, see Figure 10. The roof started to deform when the longwall face was about 10 - 32 m away, depending on the specific location of the roadway. It appears that the above distance reduced gradually from 32 m at 7 C/T to 10 m at 4 C/T. It is uncertain whether this reduction was due to stronger stress bridging effect closer to the longwall start-up or due to the possible change of roof geology and roof support design.

![Fig. 10 - Variation of roadway roof displacement against the relative distance to longwall face](image)

The roadway roof at the monitored cut-throughs stayed up until the longwall face passed the sensor locations by about 15 m in most cases. In the figure, the roof collapse was indicated as a sudden displacement drop or irregular variation when the extensometer or cable was destroyed. The process of roof deformation and collapse could only be monitored by the remote, automated system. Because the areas were inaccessible near or behind the longwall face, traditional manual monitoring methods could not be used. A typical variation of the roof displacement with depth is shown in Figure 11. It indicates that major roof delamination occurred in the vicinity of a claystone layer (C-marker) about 1 m above the roof line.

The monitored roof stresses at a selected location are plotted against its relative distance to the longwall face, see Figure 12. A rapid change in roof stress was observed when the longwall face was about 20-30 m away from the sensor location. Horizontal stresses in all the three directions (parallel, perpendicular and 45° to the roadway axis of a horizontal plane) dropped rapidly (within 1 - 2 hours) prior to the collapse of the roof. Similar stress drops have been observed in the post peak stage in laboratory compression tests. At this stage, the load bearing capacity of the roof rock was reducing while the deformation was increasing.
The pillar stress

The variation of the monitored stresses in the coal pillar as the distance to longwall face changes are shown in Figure 13. The vertical stress in the pillar showed a classic pattern. As the longwall face approached the sensor location from 400 m to 90 m, the vertical stress at the edge of the pillar (2 m into pillar) increased gradually with a peak of about 1.3 MPa at 90 m. When the longwall face progressed closer than 90 m, the vertical stress at the edge of the pillar started to decrease, indicating yield might have occurred at least within the outer 2 m of the pillar. The pillar stress in the inner part of the pillar, however, started to increase rapidly (see Figure. 13) while the stresses at the edge decreased. This was probably caused by stress redistribution after part of the pillar had yielded. When the longwall face was far away, the monitored pillar was largely intact. Therefore, the stress at the edge of the pillar is higher than in the centre. When the longwall face aligned to the pillar, the pillar yielded partially and the resultant stress in the pillar centre was higher than at the edge.

Fig. 11 - Variation of roof displacement with depth into roof at 4 C/T. Results at different longwall face positions are shown.

Fig. 12 - Variation of roadway roof stresses with the distance to longwall face
Integrated monitoring of roof behaviour

The roadway roof at 4 C/T was monitored comprehensively by extensometers, stress meters and seismic sensors. The integrated monitoring results from all the three monitoring devices showed many exciting insights of the roof failure development process.

Figure 14 shows the monitored roof displacement, roof stress change, and seismic events count at 4 C/T when the longwall face approached and passed the monitored location.

During 9/06/2004, 0:00 – 10/06/2004 12:00 (the longwall face distance = 30 m - 10 m), neither the roof extensometer nor the stress meter recorded any significant change in roof displacement and stress. The seismic system, however, recorded a major increase in the seismic events. The event count peaked periodically every 1.5 hours, coincident with the longwall advancing cycle. The longwall support was moved forward 1.2 m in every 1.5 hours after each shear cycle, leaving the roof behind the chocks to cave.

During 10/06/2004 12:00 – 11/06/2004 0:00 (the longwall face distance = 10 m – 0 m), the roof stress showed a major increase then decrease, while the roof displacement started to increase. The seismic events however, started to show a decrease during this period, before two of the five sensors failed as they were damaged or destroyed by the rock movements.

During 11/06/2004 0:00 – 12/06/2004 12:00 (the longwall face passed 4 C/T by 0 - 10 m), the roof displacement increased significantly to about 200 mm, while the stresses showed a major decrease. The remaining seismic sensors were destroyed during this time, so it is impossible to say anything about seismicity levels.

The above results suggest that in the early stage of the roof failure, seismicity is more active than the stresses or displacement. However, in the later stages of the roof failure, the stress change and roof displacement become more obvious while seismicity decreased.

The monitoring results provided a better understanding the roof behaviour during failure (Figure 15). In the early stage of the failure, roof rock under increased stresses started to fracture or delaminate at a local scale. Fracturing of roof rock caused many seismic events. But the rock was confined and remained relatively intact and hence there was insignificant change in the roof stresses and displacement.

At the later stage of the failure, local fractures started to coalesce and form large fractures or failure planes. At this stage roof stresses changed significantly and roof displacement increased rapidly. However, the seismic activities reduced since the rock mass had already released its elastic energy.
The monitoring results highlighted some issues with the traditional displacement-only monitoring. By the time the extensometers have shown significant roof movement, a major roof damage had already occurred and any remedial reinforcement measures could be less effective and more costly.

Seismicity monitoring and stress monitoring are effective in forecasting the early damage of the roof. They should be used together with the displacement monitoring to provide an “early” warning of imminent roof failure. Since they can pick up the early sign of any upcoming roof failure, roof reinforcement installed during this stage will be much more effective and less costly.

**Seismic event locations**

Over 300 seismic events were located in the vicinity of the monitored roadway roof at 4 C/T. The results are shown in Figure 16. The seismic events illustrate several failure planes in the plan view and the cross section views. The interpreted failure plane in the plan view appears to represent a damage zone developed ahead of the longwall face. In the section views, the horizontal failure planes coincide with the interface of the roof coal and rock at a height of 5 m, whereas the inclined failure planes close to the pillar could represent the breakage line of a beam due to the longwall caving.
Seismicity, failures, but the timing and intensity of each stage will vary according to site specific conditions. Seismicity and roof fall precursors and patterns have been observed at Ulan. Roof falls were observed to follow the sequence: stress change → seismicity → displacement change. This process is believed to be applicable to most roof fall failures, but the timing and intensity of each stage will vary according to site specific conditions. Seismicity and roof stress signals appear to provide warnings for the imminent roof falls earlier than the roof displacement signals.

The first field monitoring trial of CSIRO integrated real-time monitoring system at Ulan Mine has provided many interesting results. For the first time in Australia, an integrated monitoring system has been used to monitor underground roadway stability remotely and in real time. The integrated monitoring system successfully monitored the longwall initial caving. It has also provided a large amount of data on roof deformation, stress change and seismicity, which has improved our understanding of roof behaviour and failure processes.

Roof fall precursors and patterns have been observed at Ulan. Roof falls were observed to follow the sequence: seismicity, → stress change → displacement change. This process is believed to be applicable to most roof fall failures, but the timing and intensity of each stage will vary according to site specific conditions. Seismicity and roof stress signals appear to provide warnings for the imminent roof falls earlier than the roof displacement signals.

The monitoring results have the following implications to the mine operations:

- Roadway roof reinforcement should be installed in the roof loading stage, i.e. 30 m ahead of LW face. Otherwise damage may have already occurred.
- Displacement monitoring alone may not be adequate for early roof fall warning, because roof damage is likely to have occurred by the time extensometers show any noticeable deformation.

**SUMMARY AND CONCLUSIONS**

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**Fig. 16 - Seismic event locations of the first field trial of CSIRO real-time monitoring system**
• Stress and seismic monitoring can reliably warn of the caving events. They could be considered for routine monitoring.

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REFERENCE

A NEW REAL TIME PERSONAL RESPIRABLE DUST MONITOR

A.D.S. Gillies¹ and H.W. Wu¹

ABSTRACT: A new personal respirable dust monitor developed by Thermo Electron Corporation under a project funded by the US National Institute of Safety and Health (NIOSH) has generated promising results in underground coal mine testing performed in the US recently. An Australian Coal Association Research Project funded study has been undertaken to evaluate this new real-time dust monitor for personal respirable dust evaluation use particularly in engineering studies. It is believed to be the first personal dust monitor instrument (PDM) for use on mine faces that reliably delivers a near-real-time reading. It can quickly highlight high dust situations and allow the situation to be corrected.

The instrument has been tested for robustness and potential to be used as an engineering tool to evaluate the effectiveness of dust control strategies. This project has evaluated the ability of the new PDM to quickly and accurately measure changes to longwall and development section dust levels at manned points after implementation of changes and improvements. Extensive tests have been undertaken at a number of Australian longwall underground mines.

The technology that forms the heart of the personal 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 achieves microgram-level mass resolution even in the hostile mine environment, and reports dust loading data on a continuous basis. Using the device, miners and mine operators have the ability to view both the cumulative and projected end-of-shift mass concentration values, as well as a short-term five minute short term running averages. It is believed to be the first personal dust monitor instrument that reliably delivers a near-real-time reading.

INTRODUCTION

A new personal respirable dust monitor developed by the company Rupprecht and Patashnick (now Thermo Electron) in the US under a project funded by National Institute of Safety and Health (NIOSH) has generated promising results in underground coal mine testing performed in the US recently (Volkwein et al, 2004a and 2004b). Results from an Australian Coal Association Research Project (ACARP) funded study undertaken to evaluate this new real-time dust monitor for personal respirable dust evaluation particularly in engineering studies have been described by Gillies and Wu, 2005, Gillies, 2005 and Gillies and Wu, 2006.

This paper describes some results from mine studies that have been undertaken using the real-time personal dust monitor (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 achieves microgram level mass resolution even in the hostile mine environment, and reports dust loading data on a continuous basis. Using the device, miners and mine operators have the ability to view both the cumulative and projected end-of-shift mass concentration values, as well as a short-term 5, 15 or 30 minute running average. 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 has many advantages over instruments that measure from a fixed-point location. It delivers a near-real-time reading and can quickly highlight high dust situations and allow the situation to be corrected.

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An ACARP supported research project completed by one of the authors, (Gillies, 2001) entitled “Dust Measurement and Control in Thick Seam Mining” ACARP C9002 highlighted some areas for new approaches and research to allow improvement of dust conditions within extraction panels within Australia’s emerging thick seam coal industry. Industry, management, technical engineering staff and the workforce all give strong recognition to the challenge of dust as an increasing hazard particularly as higher production levels are achieved.

The underground workplace in both a continuous miner and longwall face environment has varying respirable dust conditions due to aspects such as ventilation conditions and air velocity, shearer activity and design, chock movement, AFC movement, manning position, face time of individual personnel, outbye conditions and dust levels in intake air and measurement instrument behaviour.

Many mines have observed a lack of repeatability in dust monitoring that is not easily explained. This 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.

EVALUATION OF THE PDM AS AN ENGINEERING TOOL

In the US the incidence of coal workers pneumoconiosis (CWP) has been declining for a least the past 35 years. Production levels at mines have been continually increasing and the development of dust control technologies for the working place atmosphere to protect workers has become more difficult and complex. Improved dust monitoring of coal mine dust concentrations offers a new means of protecting miners' health by more quickly identifying anomalous dust conditions.

Despite the decline in CWP, coal mine dust is still implicated in the US in the premature deaths of miners. In response, the US Secretary of Labour and the Federal Advisory Committee on the Elimination of Pneumoconiosis among Coal Mine Workers recommended that better monitoring of coal miner dust exposures be used as a method to improve miner health. In consultation with labour, industry, and government, NIOSH issued a contract to Rupprecht & Patashnick Co., Inc. (R&P), to develop a one-piece PDM. The objective of this work was to miniaturise the TEOM® technology into a form suitable for a person-wearable monitor that would enable accurate end-of-shift dust exposure information to be available to miners. Furthermore, any person-wearable dust monitor should minimize the burden to the wearer by incorporating the monitor into the mine worker’s cap lamp battery, with exposure data continually displayed during the shift to enable workers and management to react to changes in dust exposure.

The PDM is configured to provide accurate respirable dust personal exposure information in a form that is convenient to wear by a miner. Respirable dust exposure data displayed by the device has two main objectives:

- providing the miner and mine operator with timely values to avoid overexposure to dust by making any necessary changes during the course of a work shift, and
- computing an accurate end-of-shift statistic for a miner’s average respirable dust exposure.

The mass sensor in the PDM, holds the key to the accurate, time-resolved dust concentration measurements. The inertial, gravimetric-equivalent, mass measurement technique used in the device typically provides a limit of detection on par with that of the most sensitive laboratory-based microbalances. Similar to the integrated sampling method, the PDM contains a sampling system that collects particles on a filter located downstream of a respirable cyclone. In contrast to the current lapel worn personal method, however, the PDM mass measurement is performed continuously during a working shift in a mine instead of being delayed by the days or weeks required for a laboratory analysis.

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 components typically carried by the miner, while Figure 2 shows the PDM with the charging and communication module. The PDM is designed to withstand the harsh conditions found in the mine environment, with the system designed to meet MSHA intrinsic safety type approval requirements.
A 2.2 litre per minute flow of particle-laden air from the mine atmosphere enters an inlet mounted on the bill of the miner’s hard hat, and passes through conductive tubing before reaching the Higgins and Dewell (HD) cyclone at the entrance of the PDM. The sample stream with respirable particles that exits from the cyclone is then conditioned in a heated section of tubing to remove excess moisture. As the air stream subsequently passes through the mass sensor, an exchangeable filter cartridge collects the respirable particles. The mass sensor can be removed from the PDM by a mine’s dust technician (Figure 3), who changes its particle collection filter and cleans the unit after the end of each work shift.
Fig. 3 - Installing a sample filter in the mass sensor

Downstream of the mass sensor, the filtered air sample flows through an orifice used in conjunction with a differential pressure measurement to determine the volumetric flow rate. The system computer uses this information to maintain a constant volumetric sample flow by varying the speed of a DC pump.

At the heart of the TEOM mass sensor is a hollow tube called the tapered element that is clamped at its base and is free to oscillate at its narrow end (Figure 4). The exchangeable filter cartridge mounted on its narrow end collects the respirable particles contained in the air stream that pass from the entrance of the mass sensor through the tapered element. Electronic components positioned around the tapered element cause the tube to oscillate at its natural (or resonant) frequency. As additional mass collects on the sample filter, the natural oscillating frequency decreases as a direct result. This approach uses first principles of physics to determine the mass change of the filter, and is not subject to uncertainties related to particle size, colour, shape or composition.

Built-in sample conditioning to remove excess moisture minimizes the PDM’s response to airborne water droplets. The PDM determines the mass concentration of respirable dust in the mine environment by dividing the mass (as determined by the frequency change) collected on its filter over a given period of time by the volume of the air sample that passed through the system during the same time frame.

The PDM internally stores the readings from its built-in environmental sensors and mass sensor for latter downloading, and provides summary information on a continuous basis to the miner through the display located on top of the battery case. The display continuously shows the latest values for the cumulative mass concentration, the current dust concentration, and the miner’s end-of-shift projected exposure. Through this interface, miners can gauge their current dust exposure, as well as the effectiveness of actions taken to reduce the in-mine dust concentration.

Fig. 1 - Tapered element with exchangeable filter mounted on narrow end
AUSTRALIAN RESPIRABLE DUST EVALUATIONS

Two and at times three PDM units have been used simultaneously in a number of coal mines to measure conditions and to evaluate the effectiveness of dust control strategies. Since introduction to Australia in April 2005 PDMs have been used at a significant number of mines to evaluate respirable dust conditions in coal mine development sections, in longwall panels, in bord and pillar workings and outbye at points of dust interest. Data has been analysed to pin point high dust make points and allow better maintenance procedures and miner positioning to be achieved. Some examples illustrating these tests are given.

Development headings

Tests were undertaken at a development face to monitor the dust exposure levels of various equipment operators. The PDM units can give 5, 15 and 30 minute rolling averages of dust concentration. For engineering evaluation purposes it is better to use shorter time rolling average dust concentration as data the quicker response to monitored changes shows more significant dust concentration variations.

As shown in Figure 5 PDM units were put on continuous miner (CM), bolter and shuttle car (SC) operators in tests commencing at 8:15 pm. The face crew was replaced at 9:10 pm by the second crew as the first crews were released for crib break. The results of the PDM tests are shown in Figure 5 as 15 minute average dust levels.

During the tests an unplanned event took place. The end cap of ventilation ducting in an inactive adjacent face of the development section was sucked in and caused reduction in the ventilation air quantity available to the face being monitored from 7.5 m$^3$/s to 4.3 m$^3$/s. This caused a significant loss of suction head in the ventilation ducting at the face resulting in the dust-laden air at the face billowing back onto operators. All PDMs worn by the three operators have registered sharp rises in dust level. In fact this unplanned event was first noticed by one of the operators who had checked the real time display on the PDM he was wearing at the time. The failure of the end cap piece in the inactive face was soon rectified and the normal ventilation flow re-established. Readings from all PDMs show the immediate reduction in duct concentration upon rectification.

In a second test as shown in Figure 6 a development face was monitored. Two PDMs were use with one worn by the 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.

**Fig. 6 - Development Face PDM results**

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.

In a third test were undertaken at a development face to monitor the dust exposure levels of CM, bolter and SC operators equipment operators as shown in Figure 7. Ventilation at the development face was generally well maintained and dust levels appeared consistent for all face operators. Towards the end of shift, a hole through in mining the cut through from Heading A to B occurred. Ventilation at the face was disturbed when the hole through occurred and dust concentration levels experienced by CM operator, bolter and SC driver were increased.

In Figure 7, it also can be seen that before hole through, face ventilation condition was deteriorating, as the ventilation ducting was not extended during the last few metres of mining. The dust levels experienced by both CM operator and bolter, as they were standing right behind or on the machine were gradually increasing. However, dust levels experienced by the SC driver remained fairly constant before the hole-through.

During the face cut the dust readings on all PDMs increased as the distance from the end of the ventilation ducting was greater. This increase occurred consistently and was from about 0.5 to 3.0 mg/m$^3$ before hole through. A curve has been fitted to the trace to indicate that dust levels increase following exponential relationships with equation $y = 0.5666e^{0.0275x}$ and correlation coefficient of $R^2 = 0.8729$ for CM Operator and equation $y = 0.3138e^{0.0289x}$ and correlation coefficient of $R^2 = 0.8355$ for CM Operator. Fluid flow mixing relationships follow exponential relationships. Figure 8 examines in detail these extraction periods over 70 minutes with the “curve fit” relationships imposed.
Longwalls

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. Figure 9 gives a breakdown of dust make across different sources within a longwall panel. The particular LW under study ran from Chock 1 at the Main Gate (MG) to Chock 114 at the Tail Gate (TG). A number of reading sequences were taken just inbye the MG at Chock 8 or just outbye the TG at Chock 110. Dust makes for a number of measurements sequences are set down and average values calculated.
Tests were carried out as set down in Figure 10 to monitor the dust suppression efficiency of sprays in the BSL and at the belt transfer point where the longwall belt and the main trunk belt met. For the BSL test, one PDM was placed outbye of BSL, the second PDM was placed on top of the BSL inbye of the spray and the third PDM further inbye of the BSL at Chock 8. During the test, BSL sprays were on initially and then disconnected for about 30 minutes and then reconnected again. The results show that with the sprays off dust concentration levels downstream of the BSL were dramatically increased while the dust concentration level upstream of BSL remained constant with little variations.

![LW Face PDM Measurement Results](image)

<table>
<thead>
<tr>
<th>Test No</th>
<th>Chock 8 (Legs)</th>
<th>MG man</th>
<th>TG man</th>
<th>Chock man Inby Chock man</th>
<th>Chock 110 (Legs)</th>
<th>Comments</th>
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<tbody>
<tr>
<td>NS 17/10/05</td>
<td>1</td>
<td>1.00</td>
<td>1.12</td>
<td>Shadowing operators</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS 17/10/05</td>
<td>2</td>
<td>1.11</td>
<td>1.52</td>
<td>Shadowing operators</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS 17/10/05</td>
<td>3</td>
<td>3.90</td>
<td>4.57</td>
<td>Fixed position test</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 18/10/05</td>
<td>1</td>
<td>1.53</td>
<td>4.65</td>
<td>Shearer Clearer off</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 18/10/05</td>
<td>2</td>
<td>1.58</td>
<td>4.65</td>
<td>Shearer Clearer off</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 19/10/05</td>
<td>1</td>
<td>0.89</td>
<td>1.29</td>
<td>AFC dust only</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 19/10/05</td>
<td>2</td>
<td>1.12</td>
<td>1.62</td>
<td>AFC and Bank Push dust</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 19/10/05</td>
<td>4</td>
<td>1.64</td>
<td>4.26</td>
<td>AFC, Shearer &amp; Chock dust</td>
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<td></td>
</tr>
<tr>
<td>AS 19/10/05</td>
<td>6</td>
<td>1.51</td>
<td>3.18</td>
<td>Shearer &amp; Chock dusts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 20/10/05</td>
<td>1</td>
<td>1.53</td>
<td>1.29</td>
<td>Outside airstream (5 min ave)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 20/10/05</td>
<td>2</td>
<td>1.47</td>
<td>1.62</td>
<td>Outside airstream (30 min ave)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average 1.22 1.38 1.37 1.52 3.72 4.37

![LW BSL PDM Measurements](image)

**Fig. 9 - Dust make across different sources within a longwall panel**

**Fig. 10 - Dust make across a longwall BSL PDM results – 15 minute average**
It was found that the fluctuations in dust levels measured by the PDM upstream of the BSL correlated well with whether there is coal loaded on the conveyor belt or not. When there is no coal loaded on the belt the dust levels of intake air upstream of the BSL were measured at less than 0.2 mg/m$^3$. It is possible to draw a horizontal line as shown in Figure 10 to indicate whether there is coal on the belt or not.

In undertaking LW studies it is important to maintain consistency with measurement conditions along the face activities. Figure 11 indicates studies undertaken over the majority of a shift. The shearer position data was downloaded from the mine monitoring system. A cutting sequence took on average about slightly less than an hour. It can be seen in the figure that seven cutting cycles occurred across the seven hour study time period with good regularity. One early period of 45 minutes of cutting was lost to belt structure removal.

Measurements were carried out at LW face positions monitoring the dust levels experienced by shearer and chock operators in a unidirectional mining cutting sequence. Results of these tests are shown in Figure 12 to 16 for various operator position combinations.

Figure 12 illustrates monitoring dust make across the length of a shearer when cutting. One PDM 134 was worn by a person who shadowed the MG shearer operators for a cutting cycle during unidirectional cutting. The other PDM 139 was worn shadowing the TG operator. The shearer position data was downloaded from the mine monitoring system and indicated that the shearer was cutting from MG to TG first and then cutting from TG back to MG during the test. The results showed the increase in dust exposure faced by the TG operator over the MG operator. The unusual anomalous “bump” in the PDM 139 result trace at about 15:45 is put down to a significant face-slabbing fall the significance of which was very obvious to those nearby.

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**Fig. 11 - LW face dust surveys shearer position and dust monitored points m Levels**
Figure 13 illustrates dust exposure at the MG Shearer operator and TG Chock operator positions as the cutting sequences moves along the LW face. This shows under Unidi cutting that during the TG to MG cutting sequence operators are advancing chocks downstream of the shearer and so experience relatively high dust exposures. After snaking at the MG end chock operators following the shear, are upstream of the shearer and so experience relatively lower dust exposures. The results indicate that the MG shearer operator was subjected to relatively high dust level exposure when cutting from MG to TG. When cutting from TG to MG the dust level experienced by the MG shearer operator was much lower.
Figure 14 illustrates dust exposure variation in manned positions along the length of the face. PDM 134 monitored at chock 8 stationary position while PDM shadowed the TG Shearer operator during MG to TG cutting and the Chock operator position TG to MG as the cutting sequences moved along the LW face. This again shows under UniDi cutting that during the TG to MG cutting sequence operators are advancing chocks downstream of the shearer and so experience relatively high dust exposures. The PDM 139 trace shows this increasing dust exposure level which can be seen to allow some interpretation as fitting an exponential relationship. The figure shows that integration under the two position curves gives the difference in dust make which equates to total from AFC, Shearer and Chock operation. This has been calculated as 2.62 mg/m$^3$ for this test. Figure 15 shows the relative dust make experienced at the MG end of the face and the operator position closest to the TG.
Figure 16 examines whether dust make is greater during chock advance by batch (five chocks together) or individual advancement. The conclusion is that there is not significant difference.

Figure 17 examines variation of dust make with shearer advance rates. Two TG to MG cuts were examined; one taking over about 43 minutes for the cut and one only taking 27 minutes. It is clear that at the same shearer position the dust exposure of average 1.72 mg/m$^3$ for the faster cut is greater than for the slower at mg/m$^3$. 
As would be expected there is a clear relationship between these relationships and both produce a dust make of about 190 grams for each cut. This was calculated based on LW face ventilation quantity was maintained approximately at 70 m$^3$/s.

- **Fast Cut Rate:** 27 minutes and average duct concentration = 1.72 mg/m$^3$
  
  Estimated Dust Make = 70 m$^3$/s $\times$ 27 mins $\times$ 1.72 mg/m$^3$ = 195.05 g

- **Normal Cut Rate:** 43 minutes and average duct concentration = 1.03 mg/m$^3$
  
  Estimated Dust Make = 70 m$^3$/s $\times$ 43 mins $\times$ 1.03 mg/m$^3$ = 186.02 g

One comment is that dust make can be reduced by slowing of cut rate. Another observation is that dust concentration in the atmosphere at the face can be increased through increase in dilution with greater ventilation rate.

PDM tests were undertaken to examine the dust exposure levels of MG and TG shearer operators and the chockman along a longwall face during bidirectional cutting. As shown in Figure 17 it was found that when the shearer was cutting from MG to TG, both MG and TG shearer operators can experience higher dust concentration levels than when snaking at either end of the face or when cutting from TG to MG. In general the chockman experienced less dust than shearer operators during cutting as the chockman usually stands outbye of the shearer. However when snaking at the TG end the chockman may experience short periods of high exposure standing inbye of the shearer. Advances in automation of shearer cutting and chock advance and reliability of systems will influence miner positioning and exposure levels.

Figure 18 shows a significant anomalous reading which is suspected to be tied to a major goaf fall that occurred during the test. This is similar to that referred to in discussing Figure 18 tests.

![LW Face PDM Measurements](image)

**Fig. 18 - Shearer operators and chockman PDM results under bidirectional cutting**

**Belt Transfer Points**

Results of PDM tests on a belt transfer point are shown in Figure 19. Information about the tonnage on the belt during the tests was also obtained from the mine control and monitoring system. It should be noted that the tonnage was measured about 1 km away from the belt transfer point. Therefore, the tonnage on belt data was shifted horizontally along the timeline to take this into account.
It can be seen that the dust concentration measured correlates well with the amount of coal transported on the belt. The more coal transported on the belt, the higher dust concentration levels at belt transfer point.

**Air Stream Helmets**

Tests on air stream helmets were carried out at the same belt transfer point as discussed for Figure 19. Two air stream helmets were used with one worn under normal operating condition and the other worn with both the pre and main filters (as shown in Figure 20) removed. All three PDMs were used, one sampling the background atmospheric dust level and the other two sampling the air inside the two test air stream helmets. The results of the air stream helmet tests are shown in Figure 21.
An average dust concentration of 0.05 mg/m³ was measured inside the normal operating air stream helmet during the 40 minutes test period. This demonstrates that the filters used by air stream helmet can filter out most of the respirable dust. Without the filters in place, average dust concentration inside the air stream helmet was similar to that of the outside atmospheric were consistently higher than the dust levels measured in background atmosphere. A similar phenomenon was reported by others when attempting to measure dust levels inside and outside air conditional cabs (Volkwein 2005). It was concluded that an enclosed space acts as a dust trap when a jet stream injects dust laden air into a constrained space leading to higher than background dust level. In addition the jet stream in the enclosed space would keep the dust suspending longer.

Caplan et al. (1973) maintain that in air streams with velocities up to 1.5 m/s neither the air velocity nor the cyclone inlet orientation has any impact on the dust concentration measured by a sampler. However, at air velocities over 1.5 m/s, both the air velocity and the cyclone inlet orientation have an impact. Cecala et al. (1983) found that when the Dorr-Oliver cyclone inlet is pointed directly into the wind, it over samples when the air velocity exceeds 4 m/s. At very high velocities of 10 m/s it over-samples by 35 percent. When the cyclone inlet is at a right angle to the wind or pointed downwind it under-samples when the air velocity exceeds 1.5 m/s.

Cecala et al. (1983) also tested a shielded cyclone to see if a shield would reduce the over- and under-sampling. The shield was a 25 mm wide strip of aluminium sheet bent into a cylinder. This cylinder was then wrapped around the top of the cyclone and bolted to the hole in the back of the vortex finder clamp. Testing showed that the shield successfully reduced both the over- and under-sampling to within 14 percent of the true value when tested to a velocity of 10 m/s.

These evaluations were done with the traditional lapel worn personal samplers with ordinary pumps operating across the normal range of flow rates. Flow rates from these pumps are affected by conditions such as the resistance of the filter as it is loaded during sampling and hose arrangement. The pump used by the PDM has a self regulating flow rate function to correct the response to external conditions and maintains a constant flow rate throughout the measurement period. Examination of flow rates recorded in PDM data files during the air stream helmet tests showed that through out the tests the flow rates of the three PDM units remained at a constant of 2.2 litres per minutes. Therefore it should not be either over or under sampling as suggested by Cecala et al.
CONCLUSIONS AND RECOMMENDATIONS

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 and would be easy for most miners to understand.

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.

ACKNOWLEDGEMENTS

The authors acknowledge the assistance of the various mine site managers, engineers and ventilation officers who supported this ACARP project examining the PDM unit and the subsequent evaluation projects undertaken across a diversity of colliery conditions. Their efforts ensured that the principal development and mine site testing aims of the project were accomplished and a significant contribution made to future mine health and safety in Australia.

REFERENCES


OBSERVATIONS ON THE VARIATION IN ACOUSTIC EMISSIONS WITH CHANGES IN ROCK CUTTING CONDITIONS

Emma Williams\(^1\) and Paul Hagan\(^1\)

**ABSTRACT:** The emission of acoustic signals or micro seismic activity in rock subjected to stress is a well established phenomenon that has been exploited in geomechanics for example to understand changes in stress levels around excavations in active mine areas. Another potential application is in the area of rock cutting.

The finding of a study to investigate whether changes in rock cutting conditions are reflected in the nature of acoustic signals generated in rock is presented. A test facility was established comprising a linear rock cutting machine, acoustic transducer and data acquisition system. The study examined changes between a new and worn cutter pick and in depth of cut as well as the effect of attenuation of the acoustic signal with distance. The results show that there were measurable changes in the acoustic signal. Further work is suggested to expand on the range of variables considered, for example changes in rock mass type and structure.

**INTRODUCTION**

It has long been known that most solids including rock, concrete, glass, wood, metals, ceramics, plastics and ice, emit acoustic emissions or micro seismic activity (AE/MS) when subjected to stress or some form of deformation (Hardy, 1981). This phenomenon underpins the passive, indirect techniques that are currently used in industry to continuously monitor AE/MS signals of environments under normal operating conditions such as the ground surrounding active mining areas.

Machine rock cutting is a form of excavation often used in mining soft rock with machines such as longwall shearsers, continuous miners and roadheaders. Central to the design of many of these machines is a rotary cutter head around which is deployed an array of picks as shown in Figure 1. With each rotation of the cutter head, the picks in turn first impact the surface and gouge out rock as shown in Figure 2. This action sets in train transitory changes in stress levels within the rock that lead to fragmentation and the formation of discrete rock chips. As the tool continues to move though its arc of cutting there is a continual rise and fall in stress levels within the rock as chips are broken away from the surface. This cyclic pattern of loading and unloading during cutting is reflected in the variation in force on the pick as shown in Figure 3. Both the initial impact and subsequent changes in stress levels are analogous to a series of micro-seismic events.

Monitoring of AE/MS associated with the rock cutting process was first identified in the 1960’s as a possible technique that could be used as part of an integrated automated control system to guide the operation of rock cutting machines. Limitations in technology at the time however prevented further investigation.

A project was undertaken using existing facilities at the University of New South Wales (UNSW) to assess the potential of monitoring AE/MS during the rock cutting process given the advances in sensor technology and data collection. Specifically, the project attempted to determine whether AE/MS could be detected during rock cutting and whether changes in cutting conditions would translate to a change in the “signature” of the signal. This paper presents the results of this project.

\(^1\) School of Mining Engineering, The University of New South Wales
Fig. 1 - Longwall shearer drum cutting coal

Fig. 2 - Schematic arrangement of a pick during cutting (top) a rotary cutter head mounted on the boom of a continuous miner (lower). (after Roxborough and Pedroncelli, 1982)

Fig. 3 – Variation in cutting and normal forces with time during rock cutting

TEST ARRANGEMENT

The following equipment was used in the project.
• Modified Invicta 6M linear rock cutting machine with a triaxial force dynamometer attached to the cutter head.
• Standard 12.5 mm wide tungsten carbide cutting bit as used in rock cuttability tests.
• Brüel & Kjær accelerometer, model 4370.
• Brüel & Kjær charge amplifier, model 2635.
• Two 8 channel signal conditioning boards, National Instruments model SC-2043-SG.
• Two analogue to digital (A/D) cards, National Instruments model 6032E and 6034E.
• DASYLab data acquisition software system, version 7.

An experimental test facility was assembled that comprised a linear rock cutting machine and data collection system in the School of Mining Engineering and acoustic signal detection equipment from the School of Mechanical Engineering at UNSW. The cutting machine shown in Figure 4 is a modified shaping machine capable of cutting a series of grooves in sandstone up to 10 mm deep. A triaxial dynamometer attached to the cutter head measures the force on the cutting tool and resolves it into three orthogonal component vectors; that is cutting, normal and lateral force. A tungsten carbide bit was fitted to a tool post holder attached to the triaxial dynamometer on the cutter head. A schematic of the cutting arrangement is shown in Figure 5.

A sandstone block having dimensions of 395 mm (l) x 275 mm (w) x 270 mm (h) and a uniaxial compressive strength of approximately 45 MPa was secured at the front of the rock cutting machine.

The accelerometer or acoustic transducer was attached to the block of sandstone using softened bees wax. The wax which has good signal transmissions characteristics allowed for easy placement at different locations around the block. The acoustic transducer was connected to the charge amplifier by a miniature coaxial cable.

Signals from the charge amplifier and dynamometer were recorded digitally via the signal conditioning boards and A/D cards on board a PC computer. DASYLab software was used to control data acquisition during each test. Figure 6 shows a schematic arrangement of the various components in the test facility and Figure 7 shows the design of the data acquisition system (DAQ).

The DAQ was configured to record two force channels (cutting and normal forces) and the acoustic signal at a data sampling rate of 500 Hz per channel.

Following a series of calibration tests, the charge amplifier was preset to the following settings in the rock cutting tests.

• Integrator Amplifier: 316 mV/unit output
• Operating Mode : acceleration
• Lower Frequency Limit: 0.2 Hz
• Upper Frequency Limit: 10 kHz

![Fig. 4 - Linear rock cutting machine with triaxial dynamometer (left) and data acquisition system (right)](image-url)
The project involved a series of 12 tests that examined:

- cutting depth;
- state of wear of the cutting tool;
- distance between the transducer and groove being cut; and
- location of transducer with respect to the cutting direction.

Figure 8 shows the cutting tool in action during a test. The accelerometer can be seen mounted on the lower front of the sandstone block.
Fig. 8 - Rock cutting in action with acoustic transducer located at base of sandstone block

RESULTS

The results of the test program are summarised in Table 1.

Table 1 - Summary of test results

<table>
<thead>
<tr>
<th>Test</th>
<th>Depth of Cut (mm)</th>
<th>Distance cut (mm)</th>
<th>Location of accelerometer (mm)</th>
<th>Mean force (kN)</th>
<th>RMS accel (m/s²)</th>
<th>Notes</th>
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<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>233</td>
<td>X 233 Y 113 Z 135</td>
<td>new 1.027</td>
<td>0.771</td>
<td>2.142 new pick in middle of rock, transducer at end</td>
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<tr>
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<td>X 191 Y 113 Z 135</td>
<td>worn 2.114</td>
<td>2.617</td>
<td>2.022 worn pick in middle of rock, transducer at end</td>
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<td>3</td>
<td>5</td>
<td>261</td>
<td>X 261 Y 113 Z 135</td>
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<td>2.443 deep cut, transducer at end</td>
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The variation in force as measured during the first test in the program is shown in a series of three graphs in Figure 9. The top two graphs are of the cutting and normal forces. The duration of actual rock cutting was 1.67 s and the length of the groove cut in sandstone was 233 mm. Considering the sampling rate of 500 readings per second, 835 readings were recorded during the test at an average of 3.5 readings per millimetre.

The third graph in Figure 9 shows the trend in cutting forces as represented by the moving average with a period of 20 samples. This graph more clearly indicates the loading and unloading cycles of which there were approximately 10 in the test. This corresponds to the force loading cycle having a period of approximately 0.17 s applied every on average every 23 mm.

A graph of the variation in measured acceleration with time is shown in Figure 10. The graph indicates four different phases in the acoustic signal (indicated as 1, 2, 3, and 5) could be detected during a test these being:

- State 1: background noise
- State 3: electric drive motor started (State 2) and drive gear engaged, cutter head begins to move
- State 4: cutter bit impacts the rock surface and rock cutting takes place
- State 5: cutting finished, drive disengaged and power to electric motor turned off

![Graphs showing force levels and acceleration](image)

Fig. 9 - Variation in force levels during cutting Test No. 1, Cutting Force (top), Normal Force (middle)
Figure 10 - Variation in acceleration levels during cutting in Test No. 1

Figure 11 is a graph combining the calculated moving averages for cutting force with those for acceleration. While the graph shows rapid changes for both parameters, there does not appear to be any meaningful correlation between the two parameters. A closer examination over a shorter duration of just 0.3 s in Figure 12 more clearly shows some correlation in terms of the peaks in cutting force corresponding to those for acceleration.

Figure 11 - Superposition of Cutting Force and Acceleration levels in Test No.1

Figure 12 - Superposition of Cutting Force and Acceleration levels over first 0.3 s of cutting
SIGNAL ANALYSIS

Several methods of analysis of the accelerometer values were examined to quantify the effects of changes in rock cutting conditions on the nature of the measured acoustic signal. These analysis techniques included Fast Fourier Transform (FFT) and root mean square (RMS) analysis.

FFT is particularly useful when analysing irregular signals as they provide a way of isolating characteristic frequencies and quantifying the signals. A limitation of the technique is the Nyquist criterion that states a reliable frequency spectra can only be produced for frequencies that are less than half of the sampling frequency (ME 82, 2003). As the sampling rate in the test program was 500 readings per second, analysis was limited to a folding frequency of 250 Hz. Figure 13 shows the frequency spectra produced for Test 1 by FFT analysis. The rise in the frequency spectra at frequencies approaching the folding frequency of 250 Hz indicates the likelihood of frequency content at higher frequencies than the analysis can identify. Therefore the frequency spectrum is not reliable as many of the frequencies featured are probably the result of aliasing. Hence due to the relatively low sampling rate, analysis of the test results in this program using this technique could not produce meaningful results.

![Fig. 13 - FFT Analysis of acceleration levels in Test No. 1](image)

The RMS analysis of the data from each test provides a measure of the magnitude of the acoustic signal. A calibration factor was applied to the results based on an earlier calibration test using a standard signal generator.

CHANGES IN CUTTING CONDITIONS

The mean, median, standard deviation and 95 % peak values were calculated from the voltage signal and the force calibrated factor applied. The values for mean cutting force and normal force are shown for each test in Figure 14.

![Fig. 14 - Comparison of mean force levels between experiments](image)
The results show that in tests 2 and 3 when a worn cutter bit was used, the forces increased significantly especially the normal force as compared to the forces with a new bit. Conversely, the RMS acoustic value for test 2 and particularly test 3 were less than those observed with a new bit as shown in Figure 15.

![Fig. 15 - Variation in RMS Acceleration for each test](image)

The effect of state of wear on cutting force and accelerometer is summarised in Figure 16 with a doubling in cutting force and a halving in RMS of the acoustic value. A possible explanation for this difference in reaction between force and acoustic signal is that although the cutting forces are larger with a worn bit, the reduction in forces on rock fracture is much lower than with the new cutting bit. The worn bit resulted in a more irregular fracture pattern in the rock with generally smaller rock fragments. It is possible that the more frequent yet smaller fractures when using the worn bit, resulted in more acoustic activity being generated but with an overall lower amplitude. This would translate into a lower RMS acoustic value with the worn bit.

![Fig. 16 - Effect of state of cutter tool wear on Cutting force and RMS acceleration level](image)

In terms of the effect of a change in cutting depth, Figure 14 shows a doubling in depth with tests 13 and 14 resulted in a significant rise in cutting force whereas Figure 15 indicates a less significant change in the RMS acoustic value. The effect of depth on cutting force and RMS acoustic value is summarised in Figure 17 with an 87% increase in cutting force and a smaller 15% increase in the RMS acoustic signal.

![Fig. 17 - Effect of depth on cutting force and RMS acceleration level](image)
The increase in the RMS acoustic signal could be due to the larger forces required to fracture the rock causing greater amounts of elastic strain energy to be released on failure. It is this elastic strain energy that is responsible for the majority of the signals detected by the accelerometer.

In terms of the location of the acoustic transducer with respect to the line of cutting:

- when the accelerometer was placed at the front of the sandstone block, the RMS acoustic value decreased with distance;
- when the accelerometer was placed at the end of the block, there was no obvious link between distance and the RMS acoustic value.

As Figure 18 indicates, location of the transducer did not appear to have any significant affect on the acoustic signal.

![RMS Acceleration vs Distance](image)

**Fig. 18** - Variation in RMS acceleration with distance to transducer and trend/moving average of cutting force (lower).

### CONCLUSIONS

1. The test program was successful in that it showed that an acoustic signal is generated during the process of rock cutting and that it is technically feasible to measure this acoustic signal.
2. The acoustic signal was found to vary with time and with the level of forces during rock cutting.
3. The results indicate that variations in rock cutting conditions have some measurable impact of the nature of the acoustic signal.
4. Several methods of data analysis were investigated and an understanding of the theory and principles associated with each were explored.
5. The project has shown that signal analysis is a key factor to its implementation of a measurement system in the field. Better analysis methods that can be undertaken in real-time will be required if this is to be developed into a system that will be integrated into a machine control system.
6. While the project was successful in detecting an acoustic signal, the test program was limited to using an available transducer in the Faculty of Engineering. Other types of transducer are available with greater sensitivity and capable of achieving higher sampling rates than that used in this program. A study on AE/MS in coal cutting indicated a monitoring system that can handle frequencies in the range of 100 kHz to 450 kHz is required (Hardy and Shen, 1996). A resonant-type transducer was used which was highly sensitive in this frequency range.
7. With better instrumentation and signal analysis there is scope to investigate additional variables associated with field conditions, including differences in rock masses and structural boundaries; wave attenuation; changing wave velocities; changes in operating conditions, including cutting speed and force; additional acoustic background activity (including electrical interference, traffic, blasting and low level seismic activity) and wave complexities due to boundaries and rock structures.

### ACKNOWLEDGEMENTS

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REFERENCES


ME 82, 2003. Fourier Transforms, DFT’s and FFT’s – Mechanical Engineering Measurements, Department of Mechanical and Nuclear Engineering. August 20 2005 (Pennsylvania State University, USA, accessed online http://www.me.psu.edu/me82/Learning/FFT/FFT.html.)

METHODS OF INTERPRETING GROUND STRESS BASED ON UNDERGROUND STRESS MEASUREMENTS AND NUMERICAL MODELLING

J.A. Nemcik¹, W.J. Gale¹, M.W. Fabjanczyk¹

ABSTRACT: This paper presents several new methods to help interpretation and understanding of ground stress. The methods are based on data from 239 stress measurements conducted in the virgin ground in NSW and Queensland mines and computational models simulating large scale faulted ground behaviour.

The underground stress regime plays an important role in mining profitability and safety however, understanding of the stress tensor is often difficult due to its mathematical complexities and non-intuitive behaviour. The aim of this study is to explain stress distribution in faulted ground, its origin and propose several methods of stress interpretation.

Major findings presented in this study include: increase of maximum horizontal stress with depth based on underground measurements and numerical simulation of faulted ground, affect of faults on ground stress, normalisation technique that allows comparison of lateral stress magnitudes in rock of different stiffness, ‘Strain Tectonic Factor’ concept and its value in understanding stress components and its affect on rock strength.

INTRODUCTION

To date, SCT has conducted some 434 successful underground stress measurements in Australian and overseas mines. From these, 353 measurements were conducted in Australian mines and 239 tests measured pre-mining stress conditions. All stress measurements used the overcoring method of three-dimensional stress determination predominantly using the ANZI stress cell (Mills, 1997.)

The large sample of test data presented here provides an ideal opportunity to assess the in situ stress behaviour in faulted strata. This paper includes summary of the stress measurement data, methods to interpret these measurements and attempts to explain stress distribution within the tectonically strained faulted ground. The underground stress levels are sensitive to parameters such as rock stiffness, geological discontinuities, pore water pressure and gas desorption. These parameters need to be considered as they can significantly influence the measured stress in different locations and rock types. Some of these parameters are addressed here to provide understanding how they influence stress flow in rock and what methods can be used for the correct data interpretation.

To explain one of the possible mechanisms responsible for high lateral stress underground, tectonic movement of faulted strata was modelled using Universal Distinct Element Code (UDEC), (Itasca, 1999) and Fast Lagrangian Analysis of Continua (FLAC), (Itasca, 1993). The range of results obtained from the models is compared to the measured stress field underground.

INFLUENCE OF STRATA STIFFNESS ON STRESS

The vertical stress is driven by the gravitational load of the overburden strata. Horizontally bedded strata of different stiffness compress fully until they are able to carry the full overburden weight. The vertical stress will therefore be the same in all types of rock or coal strata. On the other hand, a large portion of the regional lateral compressive stress is usually of the tectonic origin caused by the movement of the Earth’s crust. In the horizontally bedded strata, stiffer rock would attract more of a tectonic lateral stress than strata of a low stiffness. The principle of stress distribution in materials of variable stiffness is illustrated in Figure 1.

¹ SCT Operations Pty Ltd
In many cases the maximum compressive stress in rock strata is expected to be horizontal and oriented in directions typical to the region. Experience indicates that rock stiffness and therefore the measured lateral stress magnitudes vary considerably in stratified roofs. To compare stress levels between two sites, stresses in rock of the same stiffness must be known. It would be impractical to look for rocks of similar properties during the measurements and therefore a 'normalising' (scaling) technique was developed to calculate stress in rock of any stiffness.

NORMALISING STRESS TENSOR

Three principal stresses \( \sigma_1, \sigma_2, \sigma_3 \) describe the three-dimensional stress tensor oriented in the unique direction at which all shear stresses are equal to zero (Herget, 1988). A change in magnitude of any principal stress would influence other principal stresses via the Poisson’s Ratio (\( \nu \)). The vertical stress in continuous bedded strata would be the same in all types of rock while the lateral stress would vary with rock stiffness. When scaling the three-dimensional stress tensor to a rock of different stiffness, the vertical stress must remain the same while the lateral stress components would change.

The gravity driven vertical stress (\( \sigma_v \)) induces a lateral compressive stress in strata equal to \( \sigma_v \nu / (1 - \nu) \) (Goodman, 1989). Assuming that the in situ Poisson’s Ratio (\( \nu \)) is similar in most rock types ranging 0.2-0.3 in value, the gravity induced lateral stress within the adjacent rock beds will range from 0.25 to 0.42 times the vertical stress. However, the in situ stress measurements indicate that the lateral stress magnitudes are in most cases much larger than the gravity induced lateral stress with a typical range from 1.5 to 4 times the vertical stress depending on location and the overburden depth. In virgin ground the ‘excess’ lateral stress is usually of a tectonic origin (Herget, 1988) and proportional to the rock stiffness (see Figure 1). The tectonic stress component determined from measurements will be dependent on large scale tectonic loading, geological structure, lithology and hydrology.

**Fig 1: Variation of stresses in different layers.**
To normalise (scale) the lateral stresses to a chosen rock stiffness, the ‘tectonic’ component of lateral stress is multiplied by the ratio of Young’s Modulus of chosen and measured rock stiffness. To summarise the normalising process:

- Choose a convenient Young’s Modulus to normalise the lateral stress into.
- Subtract the gravity induced lateral stress component from the measured lateral stress to obtain the ‘tectonic’ portion of lateral stress.
- Multiply the ‘tectonic’ lateral stress with the ratio of Young’s modulae ($E_{\text{normalised}}/E_{\text{measured}}$).
- Add the newly calculated ‘tectonic’ lateral stress to the gravity induced lateral stress component.

The ‘Normalising’ process is summarised in the equation below:

$$\sigma_{NL} = E_{N}/E_{M} \{ \sigma_{ML} - \sigma_{v} \nu/(1-\nu) \} + \sigma_{v} \nu/(1-\nu)$$

Where:
- $\sigma_{NL}$ = Normalised Lateral stress
- $E_{N}/E_{M}$ = Ratio of Normalised and Measured Young’s Modulae
- $\sigma_{ML}$ = Measured Lateral stress
- $\sigma_{v}$ = Measured Vertical stress
- $\nu$ = Poisson’s Ratio

Consider a hypothetical case where the overcore stress measurements were conducted at two underground sites. At a depth of 290 m a maximum compressive lateral stress of 19 MPa was measured in siltstone with elastic modulus of 24 GPa while at a depth of 400 m the maximum compressive lateral stress equal to 18 MPa was measured in sandstone with Young’s Modulus of 15 GPa. The lateral stress at 290 m depth was scaled down to what it would have been if the measurement was conducted in rock with elastic modulus of 15 GPa. Calculations indicate that the normalised (scaled) maximum lateral stress at a 290 m depth is 13 MPa, 5 MPa lower than at a depth of 400 m. The higher lateral stress at 400 m depth is consistent with the increase in overburden depth.

Figure 2 below shows measured and normalised maximum lateral stresses versus the overburden depth in Australian coal mines (SCT measurements only). The overall stress distribution shows no significant differences between the measured and normalised values of stress indicating a good selection of ‘average rock stiffness’ chosen for normalisation. When considering single measurements at a particular mine, the normalised lateral stress values describe the true nature of the lateral stress state at a mine site. Note that many existing discontinuities in underground mines may vary the stress flow and it is sometimes possible to experience unusual stress fields at the same depth in the same mine.

**Note:** Typically, coal has a lower stiffness than surrounding rock and therefore the maximum lateral stress in coal is usually much lower (often less than the vertical stress). Numerous overcore stress measurements in virgin coal indicate that indeed the maximum stress is in most cases the vertical stress. The stress measurements are often influenced by pore pressure loss and gas drainage within the coal that can further reduce the measured stress magnitudes in coal strata. At this stage the normalisation process is not recommended for coal due to the complex and not well understood issues affecting the stress in coal.

**INCREASE IN STRESS MAGNITUDE WITH OVERBURDEN DEPTH**

Numerous stress measurements in Australia and overseas compiled on the World Stress Map (Reinecker, 2003) indicate that the vertical and also the horizontal stresses increase with overburden depth. The normalised values of maximum lateral stress measured by SCT in NSW and Queensland coal mine roofs (Figure 3) clearly indicate increase of lateral stress with depth.
To explain the possible mechanisms of lateral stress increase with depth, several issues need to be considered. In response to a constant tectonic interaction within the ground, the rock mass on a large scale is literally broken (intercepted with many discontinuities such as faults, bedding planes, weathered dykes etc). When subject to loading, these large rock geometries would exhibit complex post failure behaviour. This behaviour can be compared to a triaxial test on broken rock sample where the maximum load ($\sigma_1$) that the rock sample is able to sustain without further failure increases with the confining stress ($\sigma_3$) applied to the sample. The triaxial test is described in Figure 4 below.

**Fig 2:** Measured and normalised lateral stresses versus overburden depth in Australian coal mines (SCT measurements only).

**Fig 3:** Increase in horizontal stress with depth in Australian coal mines as measured underground (SCT measurements only).
The exact nature of the ground behaviour may not be known, however the confining stress ($\sigma_3$) that increases with the depth of cover would provide a mechanical lock to the discontinuities within the ground rock mass. It is therefore not surprising that when laterally loaded, deeper sections of a broken rock mass would sustain larger lateral strains while near the surface where the confinement stresses are low, displacements (slips) along the discontinuities would occur more often relieving excess lateral stress until stress equilibrium is reached. The principle of this mechanism is depicted on the right hand side of Figure 3.

The stress measurement data clearly indicate that the lateral stresses measured in NSW and Queensland sedimentary strata are considerably higher than the vertical stress. These large lateral stress magnitudes and their increase with depth appear consistent with an active tectonic plate movement that would provide stress equilibrium within the ground (as discussed above).

A wide spread of lateral stress values is typically attributed to many discontinuities and non-homogeneous rock that exist within the ground. The faulted or otherwise disturbed ground can either concentrate or reduce the stress field depending on their location and depth. The probable range of lateral stress (Figure 3) versus the overburden depth can be used effectively together with geophysical logging and borehole breakout analysis (MacGregor, 2003) to estimate the probable stress at green field sites.

While substantial amount of stress measurement data has been compiled all around the world and presented in the compilation of the World Stress Map (Reinecker, 2003), SCT measurements are unique to the Bowen and Sydney Basins. The role of horizontal stress and its affect on strata behaviour in underground coal mines has been well documented (Siddall and Gale, 1992, Hebblewhite, 1997 and Mark, 2002). In most mines it can be expected that both, the vertical and the lateral stresses will increase as the mine advances to deeper ground.

**NUMERICAL SIMULATION OF LATERAL STRESS IN FAULTED STRATA**

Underground observations indicate that when the lateral stress exceeds the rock strength, low angle thrust faults form along the maximum shear planes. These planes are typically oriented at angles equal to $\pi/4 - \phi/2$ from the direction of maximum compressive stress ($\sigma_1$) (Goodman, 1989). Their cross-sections appear to be parallel to the bedding planes indicating that the maximum stress initiates the fault propagation plane in rock with similar properties and strength. During the failure, an internal angle of friction ($\phi$) in sedimentary strata would typically range between 25-35˚ indicating that a typical thrust fault in stone would dip at approximately 30˚. Any
subsequent slip along the fault planes due to ongoing tectonic movement would modify the interface properties and in general, reduce the friction along the surfaces.

A number of thrust faults were modelled using the UDEC and FLAC codes to simulate the stress equilibrium that can be sustained within the faulted ground when active tectonic displacements are applied to the model boundaries. The frictional properties along the thrust faults were varied from 5˚ to 30˚ degrees while gravity was applied to the rock mass. The results shown in Figure 5 indicate that the increase in lateral stress with overburden depth in the models were similar to the increase in lateral stress measured underground. This implies that the Sydney and the Bowen Basins are currently experiencing active tectonic compression.

As expected, the modelled results indicate that the frictional properties of fault interfaces influence the magnitudes of lateral stress that the ground can sustain during fault movement. For the fault planes with very low friction (angle of friction below 5˚) the lateral stress would be approximately hydrostatic. At 15˚ the ground appears to be able to sustain lateral stress of approximately twice the vertical stress while at 30˚ the lateral stress increases to more than three times the vertical stress (depending on the depth of cover).

Both, the modelled results and the actual underground stress measurements indicate that at the surface and at a shallow depth the ground is still able to sustain a significant portion of the lateral stress (Figure 5).

**Tectonic Factor**

The Tectonic Factor is a useful parameter that describes the amount of lateral strain induced by tectonic forces within the ground. The regional tectonic factor can be used to estimate an average ‘background’ lateral stress in undisturbed virgin ground where no discontinuities or other major structures exist.

The Tectonic Factor can be calculated by dividing the ‘excess tectonic lateral stress’ by Young’s Modulus. The calculations can be described by:

\[
TF = \frac{\sigma_1 - \sigma_v}{\nu/(1-\nu)} \times \frac{1}{E_M}
\]

Tectonic factors calculated for all SCT virgin stress measurements in Australian mines are plotted below (Figure 6).

The results indicate that the tectonic factors increase with the overburden depth. This is consistent with the higher strain equilibrium present within the deeper ground. The lateral spread of the Tectonic Factor data is attributed to the geological discontinuities and non-homogeneous rock that exist underground.

**Directions of Major Horizontal Stress**

Underground stress measurements indicate that lateral stress directions can vary substantially due to a large number of geological structures underground. In the Bowen Basin the directions of major lateral stress are in most cases confined to the North to North-East quadrant as shown in Figure 7. In NSW coalfields the maximum lateral stress directions can vary with the location and are best plotted on the regional map. Currently, other stress direction maps are being constructed in SCT to provide better understanding of the regional stress.

Variations in lateral stress direction that are sometimes measured in the mine are usually caused by at least two factors:

1. The *in situ* geological structures that can change directions of the stress flow in the mine.
2. If the lateral stresses are almost equal in all directions, the direction of maximum lateral stress can vary with even a slight change in stress.

The borehole breakout survey that is usually undertaken as part of the geophysical investigations during the exploration drilling is the best method to accurately determine the directions of maximum lateral stress flow in the explored area (MacGregor, 2003).
a) Maximum normalised lateral stress in rock versus depth for Australian mines (scaled to 15GPa rock).

b) Induced lateral stress in faulted ground driven by lateral displacements of modelled boundary (UDEC and FLAC).

Fig 5: Comparison of measured and modelled lateral stress in faulted ground.
Fig 6: Calculated tectonic factors from stress measurements in Australian coal mines (SCT measurements only).

Fig 7: Range of maximum lateral stress directions as measured underground.
CONCLUSIONS

This study presents numerous in situ virgin stress measurements conducted by SCT. The complexity of the in situ ground behaviour suggests that it may be difficult to accurately predict stress levels in the mine without actual measurements, however, a preliminary stress estimation is possible using the data presented in this paper together with other nearby stress measurements and borehole surveys.

Several important points can be deduced from this study:

- The measurements clearly indicate that in most cases, the lateral stresses are considerably higher than the vertical stress.
- An increase in lateral stress with the depth of cover can be expected in the Sydney and Bowen Basins.
- Geological discontinuities and non-homogeneous sedimentary strata can significantly influence the stress directions and magnitudes in the mine.

The data presented here strengthens the understanding of stress behaviour in underground coal mines. In response to the stress range in rock of various stiffness, normalisation (stress scaling) technique was developed that allows calculations of stress in rock of any stiffness. Recognising that a large portion of the lateral stress is probably of a tectonic origin, the tectonic factor was developed to help identify areas of highly stressed ground. Construction of stress maps showing detailed lateral stress directions in selected areas is currently in progress to help with mine layout designs.

Many geotechnical methods including numerical modelling are commonly used to predict ground behaviour. These methods require a detailed knowledge of stress distribution in the ground. A reliable source of stress information is now available to provide realistic estimates of stress in underground workings and to establish correct boundary conditions in numerical models.

A number of thrust faults were modelled using the UDEC and FLAC codes to simulate stress equilibrium that can be sustained within the faulted ground when active tectonic displacements are applied to the model boundaries. The results indicate that the increase in lateral stress with overburden depth in the models were similar to the increase in lateral stress measured underground. This study implies that the Sydney and the Bowen Basins are currently experiencing active tectonic compression.

Further research is in progress to enhance current understanding of stress and its influence on stability of underground workings in coal mines.

REFERENCES

AIRBORNE GEOPHYSICAL TECHNIQUES

Mike Armstrong¹ and Allen Rodeghiero¹

ABSTRACT: Airborne geophysical surveys have been used extensively in the mineral exploration industry predominantly for the delineation of metalliferous deposits. Recent advances in technology and the integration of multiple geophysical data-sets including aeromagnetic, radiometric and gravity surveys can provide useful information on lithology and structure. Additionally advances in data analysis, processing and image enhancement techniques have improved the resolution of geophysical datasets so that very subtle variations in the geophysical responses can be identified.

High resolution aeromagnetic and radiometric surveys have recently been acquired over the Dendrobium mine district in the Illawarra Coal Fields. The primary objective of this survey was to delineate any igneous intrusive deposits that may impact on underground Longwall mining operations. A range of enhancement techniques were also applied to the data to improve resolution and delineate the lateral distribution of sills at depth as well as anomalous features associated vertical/sub-vertical dyke deposits that may or may not reach the surface.

Airborne gravity gradiometer technology has also been successfully used to explore for a range of ore types (iron ore, kimberlites) and for geological mapping. BHP Billiton has successfully demonstrated that the FALCON airborne gravity gradiometer (AGG) can be used over sedimentary basin environments and has detected deep channels in the Gippsland Basin. A survey in the Latrobe Valley successfully delineated the coal horizons (Rose, 2005). Detailed modelling has been completed to determine whether airborne gravity techniques would be useful, specifically for delineating igneous sills at depth in the Illawarra Coal Fields. Some doubts have been raised on the suitability of this technique due to the nature of the topography, depth of cover and the lack of density contrast between intersected sills and the host sequences.

INTRODUCTION

A high resolution helicopter aeromagnetic and radiometric survey of the Dendrobium project area was flown by Fugro Airborne Surveys in April – May 2005 and acquired a total of 4461 line kilometers of geophysical data. The survey was designed to optimise the measurement of the magnetic response from igneous intrusions and structures that could influence future mining of the Wongawilli seam. With a nominal terrain clearance of 40 metres and 25 metre line spacing, the survey produced a high quality data set for the interpretation of the geological features.

In general the magnetic susceptibility or magnetic response of igneous deposits is relatively high in comparison with the sedimentary geological formations in the Sydney Basin. This is due to their content of highly magnetic minerals and in some cases such as the Cordeaux Crinanite (Teschenite) the susceptibility is a few orders of magnitude greater than background. The anomalous feature associated with this deposit is clearly visible in the lower right of Figure 1 and magnetic data from the aeromagnetic survey can be used to map its approximate surface distribution. Other igneous intrusive deposits such as dykes, pipes and sill deposits have comparatively much more subtle magnetic responses and are difficult to delineate from the host geology. In addition the magnetic susceptibilities of field samples from various known deposits in the area have very low magnetic responses and are difficult to distinguish from the host sedimentary sequences. The high magnetic response of the power lines in the centre of Figure 1 should also be noted.

Results from this survey were successfully used to target and define a number of dykes in the area and subsequently resulted in modifications to the proposed mine plan (PMP). This discussion intends to outline the various airborne geophysical techniques used in the Illawarra Coalfields and provides a case study of the recently acquired aeromagnetic and radiometric survey and how the results were used successfully to target and intersect interpreted features with inseam drilling techniques.

¹ BHP Billiton, Illawarra Coal
Airborne magnetic and radiometric surveys have been used extensively in the mineral exploration industry predominantly for the delineation of metalliferous deposits. Recent advances in technology have substantially increased the accuracy and resolution of these techniques so that they can be used to provide useful information on lithology and structure in the coal mining industry. Additionally, advances in data analysis, processing and image enhancement techniques have improved the resolution of geophysical datasets further so that very subtle variations in the geophysical responses can be identified.

The physical principles of this method are based on taking measurements of the ambient magnetic susceptibility of the surface geology and use this data to determine the distribution of magnetic minerals and hence changes in lithology. Igneous deposits generally contain a high concentration of magnetic minerals and the aeromagnetic method was originally developed to remotely detect large subsurface deposits of minable minerals. The advantages of this method are that very large areas and difficult terrain can be surveyed remotely in short periods of time thus making it very cost effective.

Airborne radiometric surveys similarly are used to measure variations in the mineral composition of surface geology and are used to map lateral lithological changes. This method involves the measurement of naturally occurring radioactive elements that exist in rock forming minerals and soil profiles. These elements are Uranium (U), Thorium (Th) and Potassium (K), which can be found as trace elements in all rocks and decay naturally giving off gamma radiation (gamma rays). These gamma rays that are emitted can be measured by a gamma ray spectrometer which can determine the source element by its peak gamma ray energy (Telford, Geldart, Sheriff, 1990).

SURVEY ACQUISITION AND PARAMETERS

A high resolution helicopter aeromagnetic and radiometric survey of the Dendrobium mining lease area was flown by Fugro Airborne Surveys in April – May 2005 and acquired a total of 4461 line kilometres of geophysical data.
The survey was designed to optimise the measurement of the magnetic response from igneous intrusions and structures that could impact on future longwall mining in the Wongawilli seam. With a nominal terrain clearance of 40 metres and 25 metre line spacing, the survey produced a high quality data set that was used as the basis for the interpretation of the geological features (Figure 2).

Encom Technologies Pty Ltd were commissioned to provide a comprehensive interpretation of the acquired datasets and provide information on interpreted anomalous features such as dykes, pipes, faults and joints that could influence future coal mining operations within the project area. Due to the subtle magnetic responses of target geological features specialised image enhancement and processing procedures were used to improve their resolution and hence, identification and geological classification.

Approximately 120 features of interest were interpreted from the magnetic survey, including sills, pipes, dykes and joints. Although there are many features of interest, the majority of interpreted features have a low confidence rating, particularly in the joint classification where low magnetic anomaly amplitudes decrease the ability to reliably track the feature trend. High confidence features were targeted for further investigation.
Modelling of the intrusive sills, dykes and pipes was used to help understand the geological characteristics of the anomalies and assist with their identification and classification. The radiometric survey helped identify the major geological units, but did not directly detect the presence of subsurface igneous bodies.

The airborne survey was flown by a jet-ranger helicopter with the measurement equipment installed in a boom (stinger) in the front of the aircraft (Figure 3).

The survey quality was very important for maximising the chances of detecting subtle magnetic anomalies from igneous intrusion. A gamma ray spectrometer was also included with the acquisition system to help with differentiation of geological units. The survey specifications and equipment of the airborne geophysical system flown by Fugro Airborne Surveys are shown in Tables 1 and 2.

![Fig. 3 - Aircraft used for low level survey operations](image)

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</table>
ENHANCEMENT OF GEOPHYSICAL DATA

High-resolution aeromagnetic survey data represent a rich source of detailed information for mapping surface geology as well as for mapping deep tectonic structure. Traditional enhancement techniques, such as first vertical and horizontal derivatives (1VD, 1HD), analytic signal (AS), and high-pass in-line or grid filters are used in enhancing magnetic anomalies from near-surface geology. Two types of filters have been developed for the purpose of enhancing weak magnetic anomalies from near-surface sources while simultaneously enhancing low-amplitude, long-wavelength magnetic anomalies from deep-seated or regional sources. The Edge filter group highlights edges surrounding both shallow and deeper magnetic sources. The results are used to infer the location of the boundaries of magnetised lithologies. The Block filter group has the effect of transforming the data into “zones” which, similar to image classification systems, segregate anomalous zones into apparent lithological categories. Both filter groups change the textural character of a dataset and thereby facilitate interpretation of geological structures. (Shi and Butt 2004).

A suite of geo-filters were applied to the magnetic data set in an attempt to provide more contrast to geological features with subtle variations in magnetic response. Table 3 below lists the various enhancement techniques used and the target geological feature it is used to interpret. Figures 4 and 5 display the resulting enhanced images used in the interpretation.

Table 3 - Geological enhancement techniques

<table>
<thead>
<tr>
<th>Enhancement Technique</th>
<th>Geological Target</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analytic Signal</td>
<td>Intrusive pipes and major dykes</td>
</tr>
<tr>
<td>Edge Filter</td>
<td>Dykes and joint zones</td>
</tr>
<tr>
<td>First Vertical Derivative</td>
<td>Surface features, basement anomalies and sills</td>
</tr>
<tr>
<td>Tilt Filter</td>
<td>Subtle Geological boundaries</td>
</tr>
<tr>
<td>Area Filter</td>
<td>Cordeaux Crinanite, detrital material and sills</td>
</tr>
</tbody>
</table>
Fig. 4 - Various enhancements of geophysical data
Fig. 5 - Various enhancements of geophysical data
REGIONAL RESIDUAL SEPARATION OF MAGNETIC DATA

This technique was specifically designed to enhance the magnetic response of intrusive sills that occur in the coal measures. A number of anomalous zones were identified by this technique and were interpreted to be caused by the presence of intrusive sills at depth.

To create this image the regional magnetic field variation due to basement rocks at depth (greater than 2000 m) is subtracted from the TMI grid to produce a residual image of magnetic intensity caused by near surface geology. The procedure used to produce a 3D model of the regional magnetic field is subjective and low confidence interpretations of magnetic sills within the coal measures were treated with caution.

The interpreted regional magnetic surface was produced by modelling and comparison with a subset of lines across the surveyed area. A number of spheres with variable magnetic susceptibilities were placed at a depth of approximately 6 km in a 3D model. Magnetic spheres at this depth produce broad magnetic anomalies, which are placed in various locations and depths in 3D space to emulate the approximate regional magnetic field. A regional magnetic surface is created from this model and subtracted from the surveyed TMI grid to produce a residual image of the resultant magnetic response, Figure 6.

Magnetic field variations due to changes in elevation were another correction that had to be incorporated into the sill modelling. Using the digital terrain model (DTM) the variations in magnetic field due to vertical displacement were subtracted from the flight line data used to model the sills. Figure 7 displays the regional residual separation image with the interpreted sills. The level of confidence of the sill interpretation was variable and correlations to borehole data are discussed later.
The interpretations provided by Encom incorporated some experimental combinations of gamma ray spectrometer and magnetic data in an attempt to produce a more definitive image of the surface geological boundaries.

These combinations were:

- RGB Image of Potassium (K), Thorium (TH) and Uranium (U)
- RGB Image of Potassium, Thorium and Uranium Ratios
- Merged Tilt Filter and RGB Radiometric Image

Images of these combinations are shown in Figures 8 to 10. From these enhancement techniques the boundaries of major geological units could be identified more accurately using the combination of their contrasting magnetic and radiometric signatures.

The resultant mapped boundary then could compared directly with the in-house geological model constructed by a combination of borehole data, seismic survey data and surface geological mapping of outcropping lithology. Any major differences in the datasets would indicate errors in the current model and anomalous features would require further investigation.
Fig. 8 - Combination K, U and Th

Fig. 9 - Combination K, U and tilt filter
Figure 11 displays the interpreted formation boundary interpreted from the data and modelled boundary constructed from a range of datasets by BHPB. The combined enhancement technique proved to be a useful tool in accurately mapping lithology outcrop in difficult and inaccessible terrain.
GEOPHYSICAL MODELLING

A range of geophysical models were constructed to simulate the magnetic signature of a number of prominent magnetic features identified on the enhanced magnetic data. The magnetic signatures that may be generated by dykes, pipes and sills of various orientations and depths were simulated to match the observed magnetic response in the data.

The following discussion outlines the models constructed to mirror the magnetic responses of potential dykes, pipes and sills that may be present in the Dendrobium area for a more definitive recognition of their resultant magnetic signature.

Dykes

There were a number of linear features that were identified on the magnetic datasets that were interpreted as possible dykes. These anomalies had variable characteristics along their length which suggested that they may be discontinuous. The ability to trace these features along reasonable strike lengths across rugged terrain suggests that the sources have moderate depth extent and are steeply dipping (Pratt and Foss, 2005)

Due to the nature of dyke bodies, being relatively thin and near vertical, the main influence on the magnetic signal detected is generated near the surface and it would be very difficult to determine if they extended to coal seam level.

In total six linear magnetic features were examined closely and models created in attempt to describe their possible thickness and orientation. The models were developed from alternate flight lines with 50m spacing and the anomalous signature modelled by the placement of vertical and sub-vertical magnetic bodies extending to depth under the flight paths. Figures 12 and 13 display a model constructed from the most prominent linear magnetic feature identified in the survey area.

Fig. 12 - Possible dyke modelled section on TMI and DEM images
The magnetic susceptibility values used in the modelling were exaggerated to enhance the predicted magnetic response of the various igneous bodies and to develop a recognizable magnetic signature for a range of orientations, thicknesses and depths.

![Perspective view of the modelled dyke reveals a general westward dipping body](image)

Fig. 13 - Perspective view of the modelled dyke reveals a general westward dipping body

Results from the interpretation revealed a significant dyke or series of dykes that bisected the proposed mine plan in Dendrobium Area 3 in a west northwest direction. These dykes were suspected to be present in this area from previous investigations and projection from dykes intersected in earlier workings. From this data a more accurate approximation of their location was determined and drilling program designed to intersect these bodies and verify their existence. A brief case study of this project and successful results are discussed later.

Igneous Pipes

The pipes were modelled using a vertical/sub-vertical elliptic pipe magnetic body. The pipe anomalies are considered less reliable to model and interpret as they are not laterally extensive and are only intersected by a few flight lines. The anomalous expressions on each of the flight lines that intersect a relatively small vertical body vary across it giving it a recognizable bull’s eye signature.

There were a number of pipe features delineated during the interpretation but after further investigation the majority were attributed to cultural features such as steel borehole casing, drill rigs and other metalliferous man made objects.

One significant anomaly of interest was modelled to ascertain its nature and orientation. This feature was detected by a total of 12 flight lines which were modelled simultaneously using an elliptic pipe model, together with an additional negative anomaly immediately east of the pipe. This negative anomaly is associated with a gully and Wongawilli Creek. Figures 14 and 15 display the anomaly, eight central sections intersecting it and a perspective view of the model.
The pipe has been modelled to be 60 m in diameter with a magnetic susceptibility of $2 \times 10^{-2}$ SI. The location of this feature is outside the current proposed mine plan and is yet to be investigated.
SILLS

A number of sill models were constructed in an attempt to identify the signal characteristics of a sill boundary. All models were constructed from the residual separation enhanced images (Figure 16).

The characteristics of magnetic anomalies over the margins of sills are dominated by the shape and abruptness with which the sill is terminated. If the sill has an abrupt termination (eg. against a fault) then that margin is marked by a sharp magnetic anomaly which both maps the location of the sill edge and provides a means to estimate its depth. This edge anomaly is very similar in character to the anomaly you would observe if a dyke were placed at that location. If, however, the sill margin tapers then characteristics of the magnetic anomaly become less diagnostic and more problematic to interpret and model (Pratt and Foss, 2005)

Single flight lines were modelled a significant distance across the boundaries of proposed sills interpreted in the area. For the several sills interpreted different thickness, depth, boundary types and magnetic intensities were used to model the anomalous signatures obtained from the residual separation enhancement technique.

The magnetic susceptibilities of igneous sill in the modelling were exaggerated to define the changes of character of the signature for different scenarios in depth, thickness and geometry.

GEOLOGICAL INTERPRETATION AND TARGET INVESTIGATION CASE STUDY

Dyke Investigations

The interpreted geophysical dataset provided additional information supporting the presence of a series of suspected dykes that intersect coal seam formations within a proposed mine plan layout. A surface to inseam drilling program was designed to intersect vertical bodies that were thought to intrude the working section of the Wongawilli coal seam. Figure 17 displays a diagram of the surface to inseam steered drilling technique used to intersect inferred dykes that were interpreted in Dendrobium area 3.

Prior to the 2005 aeromagnetic survey the inferred dykes were projected from a combination of dykes intersect in mine workings, surface lineaments and a lower resolution aeromagnetic survey that was acquired in the 1989.
The analytical signal and edge filter enhancement techniques were used to improve the geophysical interpretations of the aeromagnetic data. Figure 18 displays images of the enhanced data sets and the associated interpretations of dyke anomalies. Please note the high magnetic readings associated with high voltage transmission lines that pass through the survey area. Using a combination of all datasets and taking into account access limitations optimum inseam drilling locations and borehole directions were designed to intersect the interpreted dykes.

In two of the inseam boreholes that were completed in this area dyke material was intersected at locations that correlate very closely to the interpreted dyke locations. Changes in penetration rate and measurement of gamma ray count while drilling aid in identifying the dykes when intersected. Nominally at these locations the drill stem is pulled back and a core sample taken of the intrusion to confirm its location and thickness. Figure 19 displays an aerial photo of the drill site locations with an overlay of the inseam drill holes (light yellow lines) with respect to the interpreted dyke locations (thick white lines). The points of intersection with the dyke are denoted by dark dots.

![Fig. 17 - Surface to inseam drilling](image1)

![Fig. 18 - Inferred dyke locations with initial PMP prior to 2005 aeromag interpretation](image2)
The results of this drilling program were very encouraging and the nature and orientation of the dykes can now be modelled more accurately. A number of conclusions can be made on the use of aeromagnetic methods and associated image enhancement techniques.

![Fig. 19 - Inseam drilling dyke intersections](image)

The drilling program has confirmed the presence of a number of dykes that would impact on Longwall mining operations and the following conclusions can be made on the geological nature of these intrusions.

- The dykes are not continuous along their linear trend,
- Are almost vertical in structure,
- Range in thickness from 1 to 10 m in thickness,
- They have high magnetic susceptibility.

**Correlations of known sill occurrences with interpreted features**

There are a numbers of igneous sills that exist in the Dendrobium area and the residual separation enhancement technique was investigated as a tool to map their lateral distribution at coal seam level. The difficulties in applying a unique geophysical method to image intrusive sills at depth are:

- In the mine lease areas, igneous sills have been deposited at different geological times and may vary greatly in composition, distribution, depth and thickness.
- Sill material with low magnetic susceptibility is difficult to differentiate from the host sedimentary sequences using magnetic geophysical techniques.
- The relatively small thickness of sills at depth with respect to the overburden thickness may not produce a strong enough source signal to be detected.
The geophysical properties of sills intersected by drilling in the area vary greatly in composition and associated
physical properties. Table 4 below outlines the magnetic susceptibility and density variations found in a number
of core samples that were intersected in the field.

<table>
<thead>
<tr>
<th>Core Sample</th>
<th>Magnetic Susceptibility (SI)</th>
<th>Density (g/cm3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cordeaux Crinanite</td>
<td>$899 \times 10^{-5}$</td>
<td>2.93</td>
</tr>
<tr>
<td>Nepheline Syenite</td>
<td>$58 \times 10^{-5}$</td>
<td>2.60</td>
</tr>
<tr>
<td>Nepheline Syenite (altered)</td>
<td>$22 \times 10^{-5}$</td>
<td>2.32</td>
</tr>
<tr>
<td>Dolerite</td>
<td>$600 \times 10^{-7}$</td>
<td>3.05</td>
</tr>
<tr>
<td>Dolerite (altered/weathered)</td>
<td>$25 \times 10^{-6}$</td>
<td>2.60</td>
</tr>
</tbody>
</table>

In total seven areas of sill were interpreted to occur in the Dendrobium Area. Some of the interpreted sills of high
confidence correlated well with borehole data and those of low confidence did not. The results confirm that the
imaging techniques for identifying possible areas of silling are limited and that igneous deposits with low
magnetic susceptibility such as the Nepheline Syenite cannot be delineated using high resolution aeromagnetic
techniques.

HIGH RESOLUTION AIRBORNE GRAVITY TECHNIQUES

Airborne gravity exploration techniques are used predominantly in regional geophysical exploration to delineate
significant geological formations and structures in both the Petroleum and Mineral exploration industries. Once
again, recent advances in technology have warranted assessment of this technique on targets with more subtle
geophysical responses.

Airborne gravity gradiometer technology has been successfully used to explore for a range of ore types (IOCG,
Iron Ore, Kimberlites) and for geological mapping. BHP Billiton has successfully demonstrated that the FALCONTM
airborne gravity gradiometer (AGG) can be used over sedimentary basin environments, and has detected deep
channels within them in a survey flown over the Gippsland basin. A survey in the Latrobe valley successfully
outlined the coal horizons (Rose, 2005).

This method is based on the measurement of changes in the Earths surface gravity due to the variations in the
densities of subsurface geology. Gravity measurements are made using an airborne gravity gradiometer (AGG)
system that was developed by BHP Billiton and Lockheed Martin.

In its simplest form, a gradiometer consists of two gravimeters. They experience the same aircraft accelerations
(e.g. reference ellipsoid, latitude, Earth tide and isostatic effects). The gradient is simply the subtraction of one
gravimeter response from the other, so most of these corrections cancel out, and we are only left with self-gradient
and terrain corrections as the key factors (FALCON®, 2005).

A trial of this technology was considered as a mechanism to assist in delineating igneous intrusions at depth,
specifically silling in the Dendrobium region as well as any major structural boundaries. To provide further
information on the viability of this method, Falcon AGG were commissioned to demonstrate that the density
contrast of igneous bodies at depth were sufficient to produce an anomalous signature.

Terrain effect

The terrain effect is an important correction that is applied to the Falcon data before an interpretation of the
subsurface density variations can be undertaken. If the terrain effect is large as it is here, and it cannot be
corrected adequately, then the errors introduced may be interpreted as having a different source. Terrain
correction is applied with a constant density, chosen to be as close as possible to the outcrop. Reasons why the
terrain correction may cause a problem include;

- The upper section being inhomogeneous, for example where incision of the topography exposes
  section with different densities
• The Digital Elevation Model (DEM) is not sufficiently accurate

The Falcon AGG system has on board a laser scanner to measure detailed DEM. To get complete coverage, the line spacing should be 130m if flying at 80m above ground. Increasing the flying height enables the flight line spacing to be increased while still acquiring complete laser scanner coverage for the DEM.

Modelling of Geophysical Responses

Igneous intrusive deposits modelled from borehole data and mine intersections were used to model their synthetic gravity response, as they were a good representation of a range of possible sources. A synthetic survey was designed to cover the area of the Dendrobium mine and then processed as it would be in a Falcon survey.

A total of six possible sills were modelled at different depths from the surface and with different thicknesses to see if they could be imaged by their contrasting apparent density to the ambient host rock. The density values used in the modelling were between 2.9 and 3.0 g/cc which are typical of igneous bodies such as the Cordeaux Crinanite and various Dolerite deposits.

The synthetic modelling showed that with normal survey conditions and noise, the igneous intrusions are able to be measured with the Falcon AGG if they are 40 metres thick, and of sufficient aerial extent. Modelling was done in the absence of other density variations, including any related to the outcropping terrain and associated terrain corrections (Rose, 2005).

In reality the thickness of the majority of sills intersected in the area are around coal seam thickness (2 – 3 m), with the major bodies ranging from 10 to 50 m thick. Additionally the highly incised nature of the terrain in this area would result in difficult and dangerous flying conditions as well as extremely complex terrain corrections that would have to be applied to the data. The results indicate that the igneous bodies would have to be of significant thickness and distribution to have a detectable response and although results are encouraging, the difficulty in directly imaging the distribution of intrusive sills in this area would be highly experimental.

CONCLUSIONS

Airborne geophysical techniques have been found to be effective in delineating surface and subsurface features and, when used in conjunction with other exploration techniques, become an important tool for developing a comprehensive geological model.

Integration of various airborne geophysical datasets is essential for successful interpretations of subtle geophysical responses. With the aid of more advanced processing, mathematical enhancement techniques and higher confidence in geophysical data, a more accurate exploration campaign can be designed to verify the presence of specific geological targets.

These techniques have been successfully used by Illawarra Coal to obtain a more detailed geological understanding of igneous intrusions and the nature of geological structures. This, in addition to and when used in conjunction with other exploration data-sets, provides an invaluable tool to assist in the development of effective mine plans.

REFERENCES

ADVANCES IN SURFACE SEISMIC ACQUISITION, PROCESSING AND INTERPRETATION

Allen Rodeghiero¹ and Luke Fredericks¹

ABSTRACT: Seismic Reflection surveys and borehole drilling have been the two primary exploration tools used in Illawarra Coal’s operations in the Southern Coalfield. For the past 10 years the exploration department has been using their in-house acquisition system developed by BHP. However, this system was limited to 180 channels and two dimensional surveys and a more advanced system was required with a much larger channel capacity for modern three dimensional seismic surveys. In July 2004 Illawarra Coal acquired a more advanced seismic acquisition system from Vibtech in the UK, which enabled high resolution three dimensional surveys to be conducted.

Acquisition, processing and interpretation techniques have also been improved through the use of three component inseam geophones, depth conversion of data, shear wave acquisition for the near surface interval, statistical analysis and integration with other data including boreholes, downhole geophysical logs, seismic, airborne magnetics, surface and inseam mapping, surface to inseam drilling and inseam drilling.

Processing and interpretation techniques have been refined to suit the local geology within the Illawarra region. Some of the methods used to improve the value of the interpreted data include:

- Depth Conversion
- Full waveform sonic and VSP to improve velocity analysis
- Acquisition, processing and interpretation of three component geophones at coal seam level to define structural lineaments, stress domains and possible dykes and
- Modelling of strata gas reservoirs from seismic and downhole geophysical data

INTRODUCTION

Seismic reflection surveys have evolved to become an integral part of resource exploration. This technique has evolved to allow access to sensitive land areas and also rugged terrain and minimising land disturbance. Since the early 1980’s seismic surveys are an integral component of exploration programs to ensure that future mining areas are free of detectable structure and hence contributing to the effectiveness of mine planning.

Illawarra Coal has recognized the value of high resolution seismic surveys, resulting in the purchase of a new acquisition system in 2004 known as the Vibtech Infinite Telemetry System with an 840 channel capacity to advance our seismic acquisition into the 3D realm.

Illawarra Coal’s mining lease areas can be subdivided into two main geographical regions. The Northern area consists of relatively undulating countryside. The region is moderately populated and consists of a number of townships, rural farmland and considerable infrastructure such as main highways, high voltage transmission lines, gas lines, water reservoirs and canals. The Bulli seam is mined in this region by Appin, Douglas and West Cliff Collieries. Within the southern area, Dendrobium Colliery mines the Wongawilli seam and is located within Sydney Catchment Authority land which consists of deeply incised gorges and rugged bush. Figure 1 shows the location of Illawarra Coal’s operations.

¹ BHP Billiton, Illawarra Coal
BHP Billiton Illawarra Coal operates in the Southern Coalfield, which is the southern portion of the Permo-Triassic Sydney Basin, as shown in Figure 2, and contains the Illawarra Coal Measures of Late Permian Age. Sandstones, shales and mudstones of the Narrabeen Group, which in turn are overlain by the Hawkesbury Sandstone, a massive quartzose sandstone unit that varies in thickness due to erosion, overlie the Illawarra Coal Measures. The Wianamatta Group lies stratigraphically above the Hawkesbury Sandstone and is the uppermost unit in the northern part of the Southern Coalfield.

Within the Illawarra Coal Measures the Bulli Coal is the uppermost coal member and is the only economic seam that is currently mined at Appin, Douglas and West Cliff Collieries. The Wongawilli Coal, some 30-40 metres below the Bulli Coal, is mined at Dendrobium colliery.
HISTORY

Seismic Reflection surveys have been conducted in the Illawarra Coal region since the early 1970’s. Throughout the years the technology and techniques have improved increasing the confidence of information obtained. The improvement of the data quality has resulted in seismic exploration methods becoming an integral part of resource calculations.

BHP Collieries developed its own in-house seismic acquisition system in the early 90’s which had a 180 channel
capacity. This limited the surveys to two dimensional lines and small low-fold 3D. Although providing high resolution the system was inhibited by slow acquisition time, reduced flexibility in surface access for acquisition and limited off-set capability. The new equipment commissioned in June 2004 is the Vibtech Infinite Telemetry System with an 840 channel capacity. This has allowed the department to undertake an extensive 3D campaign with high resolution data to accurately image relatively small faults at coal seam level. On good quality 2D data, faults are only detectable when their vertical displacement is greater than seam thickness. With good quality 3D data, image processing allows small displacements to be imaged on the coal seam reflection surface. Resolution of faults down to half seam thickness displacement is possible with this method. The new system utilises telemetry to transmit the data from the digitisers to the recording station. This allows rapid data return and acquisition in difficult terrain.

2D surveys are conducted in areas where access is limited or low impact access is required due to environmental constraints. Historically dynamite has been used as the primary energy source and is preferred. However, the majority of exploration areas are located in environmentally sensitive, residential and/or rural areas and the use of alternate source energy is currently being investigated. The use of low impact seismic sources such as Vibroseis, Mini-Vibroseis and Mini-Sose mechanically input seismic energy into the ground via heavy equipment, eliminating the need for drilling and explosives.

Early seismic surveys utilised Oil field technology with low often single fold 2D data acquired over large regions. Broadly spaced shot and receiver locations provide low resolution data quality, in excess of 40m at coal seam level. Data recorded from these early surveys were only able to identify large scale structures and coal seam continuity. There is a specific example where a fault with a 60m throw was not detected.

During the 1980’s as technology improved substantial experimentation were conducted to improve the seismic method. Various source configurations were trialled including directional shot and multiple shots. Directional shots were used to direct the source energy toward the coal seam, in theory to achieve a stronger reflection signal and better resolution of the coal seam. Multiple shots in the same hole were tested to enable a more accurate analysis of the slower velocity weathered zone in the near surface. However, these techniques were not adopted due to poor results and limited resolution.

Source and receiver configurations were tested extensively to determine the optimum parameters for reflection techniques on coal seam targets in the Southern Coalfields. This led to higher resolution closely spaced receiver and nominal source spacing used in modern survey techniques.

Good results from the various trials and experiments throughout the 80’s and early 90’s encouraged BHP Collieries to develop their own seismic acquisition system. In 1993 the Surface Seismic Portable Transient Recorder (SSPTTR) system was developed. This system consisted of a 180 channel capacity and a 24 bit system capable of recording at a 0.25 millisecond sample rate. Resolution was significantly improved resulting in a useful tool for defining any fault structures that would impact on underground coal mining operations. Vertical displacement down to 5 m could now be imaged.

The primary technique before 2004 was high resolution 2D surveys, which defined the boundaries of the current mine plans. Parameters were nominally 5 to 20 m line spacing for shot and geophone locations with 450 g of high explosive placed at the base of 14 m drill holes, which places it below the weathering zone. These shot holes are then stemmed to the surface with gravel in order to focus the explosive shockwave below the slow velocity layer and prevent venting of material. Initiation of the shots is via a coded UHF signal from an Encoder at the Central Control Unit known as the ‘Macha or Boom Box’ in which, a time break is set to synchronise the detonation of the shot with the recording system. The signal is received by a Decoder wired to the detonator at the shotpoint which detonates the source via an electrical signal.

Trials have been conducted using Vibroseis, Mini-Sose and Mini-vibe as a replacement source energy to be used in areas where drilling of shot holes and the use of dynamite are not desirable. These areas include roads, higher density residential and environmentally sensitive areas. In late 2005 Mini-Vibe trials were conducted over a 2D line that had already been acquired using the traditional dynamite source with the same acquisition parameters for a direct comparison of data quality. Figure 3 displays an image of the mini vibrator during acquisition.

The results of this trial were encouraging with reasonable data quality and reflector continuity, making it a viable option in sensitive areas. The dynamite source energy, however, has a much broader dynamic range and provides better data quality and hence higher resolution. Further trails are to be conducted later this year using a Hemi-60, 30 tonne vibrator to verify if better data quality can be obtained with a higher energy vibroseis source.
VIBTECH SEISMIC EQUIPMENT

An expression of interest by BHP Billiton Illawarra Coal for a new seismic system in 2003, resulted in Vibtech Infinite Telemetry System being selected and was commissioned in June 2004. The commissioning determined that the Vibtech system complied with BHP Billiton and Australian standards along with the operational requirements of Illawarra Coal.

The Infinite Telemetry system is a hybrid of cellular and cable communication. The cellular nature of the Remote Acquisition Units (RAU) (Figure 4) free the design and deployment restrictions previously experienced in many full cable systems. Each RAU has four channels with Illawarra Coal purchasing an 840-channel system. Central Access Nodes (CAN) (Figure 5) receive the cellular data from the RAU’s and send it via optic fibre cables to the Central Computer Unit (CCU) for recording. This provides a rapid data transfer by reducing the data traffic, particularly when operating all 840 channels.

The equipment has been designed with a comprehensive status, alarm and fault detection system, which provides the operator and maintenance personnel the facility to monitor the operation of the equipment and to quickly identify any faults. RAU’s and CAN’s have built in LED systems that give instant diagnostics of fault and operational modes. The operator at the CCU has a comprehensive remote display of the majority of the diagnostics, which are easily viewed, updated and on many occasions can be remotely remedied.

One of the advantages of using Vibtech was the ability to provide hardware and software upgrades to suit Illawarra Coal requirements. This is usually difficult with larger oil based supplier equipment. During the time of purchase the system was the lightest per channel on the market, which has benefits for BHP Billiton Health Safety Environment and Community (HSEC) standards and permits the use of a small field crew for acquisition. Lithium-ion batteries power all the field equipment which dramatically reduces equipment weight compared to traditional lead-acid batteries. Future refinements and technology advancements will enhance the equipment and improve the acquisition process.
Fig. 4 - Remote Acquisition Unit, RAU

Fig. 5 - Central Access Node, CAN
The main advantage of the new system is the capability to conduct 3D surveys. The 840 channels allows a lateral surface cover of approximately 1km$^2$ in a static layout and provides enough redundancy to roll the equipment along while shooting for unlimited size surveys.

The IT system has a high dynamic range with the ability to record down to 0.25 millisecond sample rates. Data transfer from the RAUs to CANs is wireless via a 2.4 GHz ISM band allowing rapid data transfer. Geophones are 48 Hz and are grouped into sets of 4 which are digitised by a single RAU. All RAU’s receive a GPS timing signals via a VHF radio, this ensures an accurate and unique time break for each shot.

The fibre optic, despite allowing rapid data transfer, has also become one of the major issues during acquisition. The fibre is military specification, with a tough Kevlar and plastic coating which is deployed between CANs. Animals such as kangaroos, rodents and livestock often chew through the casing cutting the fibre connection and lead to substantial costs for repairs. Future development of a 5.8 GHz radio Cell-Link between CANs will eliminate the need for the fibre. Figure 6 displays an advanced digitising unit that includes GPS timing on all RAUs. This RAU is currently in development and expected to be available in 2007. In some locations, such as dense bush and undulating terrain, the VHF signal is interrupted and it is not possible to acquire the correct timing. The GPS on each RAU would eliminate this problem. As GPS technology advances the RAU’s could also self survey, which will reduce site survey costs.

![Fig. 6 - Future RAUs with inbuilt GPS units for timing and survey](image)

**2-D AND 3-D SEISMIC METHODS**

**Survey Design**

Survey parameters used by Illawarra Coal have been refined through significant testing to optimise the quality of the data received. Two and three dimensional surveys are both widely used. Despite the push to conduct 3-D surveys with their superior resolution at coal seam level and total area coverage, 2-D lines must still be used to acquire data in less accessible and environmentally sensitive areas.
2-D Seismic Surveys

The 2-D method used by Illawarra Coal involves a line of receivers, geophones, usually greater than 2 km to provide and adequate data spread. The inline distance between receivers is 5 m-20 m, shot points are placed at every 2nd or 3rd receiver. Fold is calculated over the length of the 2-D survey and denotes the number of common reflection points or common mid points (CMP) that occur at each geographical location along the line. The higher the fold the better the resolution obtained from the data. In a typical 2-D survey design a nominal fold of 30 is required to obtain the desired resolution. As discussed earlier the quality of seismic reflection data varies from site to site and field parameters must be modified to achieve a similar quality result. Table 1 below outlines the different field parameters used in mine areas with different geological conditions.

Table 1 - Variations in field layout parameters

<table>
<thead>
<tr>
<th>Mine Area</th>
<th>Shot Spacing</th>
<th>Receiver Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appin Area 3</td>
<td>5m</td>
<td>10m</td>
</tr>
<tr>
<td>Dendrobium Area 2 &amp; 3</td>
<td>7.5m</td>
<td>15m</td>
</tr>
<tr>
<td>Douglas Area 7 &amp; 8</td>
<td>10 m</td>
<td>20m</td>
</tr>
</tbody>
</table>

Reasons for the change in data quality include;

- Changes in surface lithology – Sandstone / Shale
- Residual soil depths and weathering zone
- Nature of topography
- Residential, environmental and infrastructure obstacles.

To achieve higher fold along a 2-D line with a major obstacle, for example a highly incised gorge, additional shots are included in the design on either side. This increases the nominal CMP coverage beneath the structure resulting in better resolution and continuity of coal seam reflectors. Shot spacings are reduced at the start and end of 2-D lines to boost the fold and increase resolution on the geographical survey limits. Figure 7 displays a survey configuration of a 2-D survey acquired over an 80 m gorge and the resulting 2-D seismic section.

Fig. 7 - Increased number of shots either side of the gorge to boost fold
3D Seismic Surveys

To date there have been a total of twelve 3-D surveys acquired in the Illawarra Coal mine lease areas. Two of which were conducted prior to the commissioning of the Vibtech equipment using contract equipment. There is considerable variation in design depending on the depth and nature of the targets. Prior to the purchase of the new equipment, orthogonal geometry was used for acquisition and although full coverage was obtained there were a number of discrepancies noted in the data.

Orthogonal geometry requires the shot lines to be positioned perpendicular to the receiver lines. Figure 8 displays the acquisition footprint resulting from orthogonal survey design from the Cataract 3-D survey acquired in 1998. This occurs when offset and azimuth ranges cause a deterioration in fold in linear zones. Parallel geometry such as this is commonly used for marine surveys and requires small distances between shot and receiver lines for adequate cross-line sampling to avoid this issue.

To minimise the impact of this phenomena an alternate survey design known as slant geometry was implemented. Slant geometry is simply a method of positioning the shot and receiver lines at a discrete angle from one-another to more evenly distribute CMP locations for a smoother result. Slant geometry tends to have better distribution of offset and azimuths for low fold data (Vermeer, 2002). This results in increased data quality and reduced survey costs. This technique has been adopted for 3-D survey design with an offset of 45° found to have the best result in fold coverage and data resolution.

Calculating fold, azimuths and offsets is essential for a successful survey. High fold is necessary for suppressing the recording noise, including ground roll created by the source and background noise such as wind, roads or overhead powerlines. In a 3-D survey fold is calculated by the number of CMP traces that fall inside a 10 x 10 metre bin in the survey geometry. For optimum results surveys in the Illawarra Region are designed to achieve 20 to 30 fold using a 600 m offset limit. This has been found adequate to minimise the signal to noise ratio.

MESA Field software is used for the design process. It is a user friendly PC based software in which surveys can be designed and shot synthetically to provide optimum field parameters and fold coverage. Figure 9 is an image of the MESA software display showing an aerial photo of the survey area with nominal shot and receiver positions overlain by an image of the resulting fold distribution. This is a powerful tool for quickly and easily designing a 3-D survey with optimum field parameters for the best overall result.

Once the design is finalised, the software has the ability to produce survey scripts created from the design that can be loaded directly in to the acquisition system to enable an immediate start to the survey.
Acquisition and Logistical Challenges

The lightweight nature of the equipment has allowed a smaller crew for acquisition, with an average of one observer, one shot firer and five other field personnel. This results in constant work for the group, who are involved with the flagging, pegging of shot and receiver locations, drilling, acquisition and rehabilitation of the site.

Safety is the primary focus of many field operations. The seismic crew has had extensive training covering all aspects they will encounter in the field. There is an initial BHP Billiton induction covering all 15 HSEC standards and 10 fatal risk protocols. Individuals are then trained and tested on the entire seismic standard operating procedures as well as undertaking a 4WD training course. The crew is also involved in Leading Zero harm, BHP Billiton safety initiative, which highlights any safety risks in the field.

Logistical issues such as land access, highways and underground services can be taken into consideration in the design stage, where modifications are made to the design to negotiate a difficult area. Alternate sources such as Vibroseis, Mini-vibe and Mini-Sose have also been used as an alternate source to dynamite to minimise the environmental impact. Technology advances such as Magneseis detonators were developed as an alternative to electric detonators for shooting in close vicinity to powerlines.

The following are examples of 3-D designs that required substantial planning in difficult areas:

High voltage transmission lines and high pressure gas lines passed through a West Cliff 3-D survey area as shown in Figure 10. An exclusion zone around the gas line of 100 m was required for shot locations and only low impact vehicles, John Deer Gators, were permitted to drive over the gas line easement. Shots holes were not drilled under the power lines due to the safety reasons and where holes were required close to power lines, Magneseis detonators were used.
Magneseis detonators were adopted by Illawarra Coal in the 1990’s as safety concerns were identified that electrical detonators may be initiated by electrical currents produced by the ambient electromagnetic field associated with high voltage power lines. Magneseis detonators, as the name suggests, use a magnetic coil that produce a specific electromagnetic field to initiate the shot.

A Douglas 3-D survey shot in January 2005 faced a number of logistical issues including; the Hume Highway, a large gorge and the gas line. The telemetry nature of the equipment enabled receivers to be placed on both sides of the Hume Highway, (Figure 11) which allowed data to be collected from under the highway. In the centre of the survey there was a large gorge where shots and receivers could not be positioned. To overcome the issue, additional shots and receivers were positioned around the gorge to boost fold under it.

SEISMIC PROCESSING AND INTERPRETATION

Seismic data consists of digital samples of seismic energy recorded at each channel or geophone at discrete intervals in time series. For example in a typical 3-D seismic survey, digital data must be recorded simultaneously at 840 channel locations at a sample rate of 0.5 milliseconds. The record length for these shallow reflection seismic techniques is usually one second which equates to 1.7 million samples recorded at any one shotpoint. In a typical 3-D survey acquired by Illawarra Coal there would be anywhere up to 2000 shots recorded for the entire survey. Seismic data processing is required to sort this data into a useable format and includes the application of mathematical algorithms to boost signal and remove unwanted noise. This is required to produce two and three dimensional images of the subsurface geological structure. Reliable and accurate processing techniques are required to convert raw seismic shot records into interpretable seismic sections. Figure 12 displays the seismic energy recorded from a single shotpoint in a 3D survey.
Fig. 11 - Logistical issues of 3-D survey Douglas_8_1, Deep Gorge, Hume Highway and gas line

Fig. 12 - Typical shot record from a 3D seismic survey

This raw data record displays the seismic data recorded and the inherent noise associated with ground roll energy, noisy channels and loss of signal with depth.
Processing Flow

The processing flow applied to the seismic data is dependant on the nature of the geological subsurface at any one site and may vary greatly from location to location or changes in geological environment. As discussed in a previous section the geological environments in the Illawarra Coal exploration region are essentially subdivided into two main regions with significant differences in topography and geological overburden.

Although differences exist in velocity analysis, deconvolution and residual static calculations of various datasets the processing flow or set of processing operations remains the same for all seismic data acquired to ensure interpretation reliability across the exploration areas.

The data processing is contracted to Velseis Processing in Brisbane who apply a standard processing flow to all datasets and produce four final versions of each with different processing parameters applied. The data can then be viewed with different processing applied for improved structure interpretation. The different products supplied by Velseis include:

- Filtered and Migrated with Spectral Whitening,
- Filtered and Migrated,
- Unfiltered Final, and
- Unfiltered and Migrated Stacks.

The nominal processing sequence is as follows:

1. Reformat Promax internal format
2. Edit Bad & Reverse Polarity Traces
3. Apply geometry
4. Apply Refraction Statics
5. Correct for Spherical Divergence
6. Surface consistent Spiking Deconvolution
7. Spectral Whitening
8. 1st Pass Velocity Analysis
9. 1st Pass Residual Statics
10. 2nd Pass Velocity Analysis
11. 2nd Pass Residual Statics
12. Common Midpoint Point Trim Statics
13. Pre-stack Automatic Gain Control
15. 2D Steep Dip Explicit FD Time Migration, Smoothed Stacking Velocities
16. Post stack frequency enhancement
17. Bandpass Filter

The refined processing procedures provide two dimensional sections of the subsurface geology and the coal seam reflectors are then digitised to provide a structural surface of the coal seam. Figure 13 displays a fully processed seismic section with the Bulli seam and faults interpreted.

Interpretation and Results

The interpretation process involves viewing all seismic sections in the survey and interpreting or picking the reflected energy from the Bulli coal seam. All data can then be depth converted using another processing technique, where seam depths from borehole data and mine levels are tied in with the seismic picks to convert the seismic data from time domain into true vertical depth.

Initial interpretation of the data involves picking the Bulli seam reflector, mapping any faults and/or disturbed zones. Figure 14 is a three dimensional topographic image of the top of the Bulli seam created from digitised picks of the interpreted Bulli seam reflector from a recent 3-D survey. The picks are extracted from SEGY files as x,y,z coordinates in which there are approximately 10,000 points for every 1km² of survey data. The SEGY file data is viewed on SeisWin, software which was created by Dr Binzhong Zhou of CSIRO (Figure 13 and 15). These figures display fault structures that can be clearly identified on the sections.
Fig. 13 - 2D seismic section showing Bulli seam and interpreted faults

Fig. 14 - Bulli seam surface from a recent 3-D Seismic survey
The SeisWin Depth conversion involves picking the Bulli seam reflector and saving the picks into the seismic lines file. The picks from all the lines in an area are then tied to one another so that the Bulli reflector time on one line correlate to that on another. This creates a Bulli time surface which is in turn is converted into a depth horizon by applying a velocity correction to the time surface. Currently there are 250 2-D lines and 12 3-D surveys that have been depth converted creating good control for seam structures.

One of the main challenges facing Illawarra Coal is to predict areas where longwall mining will be hindered by small structures of seam thickness displacement. This is difficult from 2-D surveys as the resolution and line spacing of the technique often is not adequate to reliably pick the zones. Much more success has been gained from 3-D surveys where disturbed reflectors can be mapped through a zone (Figure 16).
Methane gas trapped in sandstones above the Bulli seam (known as strata gas) can be recognised on borehole geophysical logs. Recent work has been done to map the distribution of gas by identifying it in borehole and seismic data. The presence of gas in overlying strata can be recognised on 2-D and 3-D seismic sections due to the attenuation of seismic energy with depth. Gas attenuates the seismic signal and results in the loss of reflected energy below the gassy zone. See Figure 17.

Seismic attribute mapping a technique used by the petroleum industry is currently being investigated as possible tool for mapping the lateral distribution of strata gas.

Igneous intrusions are usually mapped by their magnetic susceptibility through aeromagnetic surveys. Some of the intrusions in the Southern Coalfield, such as the Nepheline syenite have low magnetic responses similar to the sedimentary host rocks and cannot be identified using this technique. In these areas seismic has been utilised to identify jacking between the coal seams due to sill intrusions. When deposited, igneous material intrudes weak bedding zones and coal seams destroying the coal quality and lifting up overlying sequences. Figure 18 displays a seismic section which has confirmed the presence of a known sill by the changes evident in strata thickness and loss of seismic signal below.
VERTICAL SEISMIC PROFILE (VSP)

Vertical Seismic Profiling, VSP, is the process of recording seismic energy in geophones placed down a borehole. Illawarra Coal has been installing tri-axial three-component geophones, two horizontal and one vertical, at the Bulli seam in all of their coal quality holes for the past 20 years, in some instances a geophone string with 60 channels down the length of the borehole to the Bulli seam. The new seismic digitisers are easily attached to the borehole geophones and are incorporated into 2-D and 3-D surveys providing an additional data-set.

The VSP is where the compression P-waves and shear S-waves are analysed to determine geological conditions such as stress fields and vertical structures. P-waves travel faster along the primary horizontal stress direction and slow when the pass through vertical fractures (Figure 19). When the S-wave passes through a fracture it splits into two separate components, SV-wave (vertical) and SH-wave (horizontal), the ratio between the two give an indication of the location and orientation of the vertical structures (Figure 20).

Fig. 18 - Jacking between the Bulli and Wongawilli seam reflectors due to a sill

Fig. 19 - The effect of stress on the orientation of maximum P-wave velocity (Sato, 2005)
The principal stress direction is very important for effective mine planning as the orientation of the longwall must be parallel to the stress direction to minimise strata control problems. The stress direction is determined by analysing the arrival times of primary (compressional) seismic energy from each shotpoint on a survey. When the velocities are plotted in plan view a linear trend of high velocities becomes apparent and the principle stress direction determined. Figure 21 displays the principal stress direction determined by breakout analysis from the borehole acoustic scanner which is coincident with the maximum velocity and interpreted direction of maximum horizontal stress from the VSP analysis.
CONCLUSIONS

Advances in seismic acquisition systems, survey techniques, data processing and data analysis have been utilised by Illawarra Coal to significantly increase confidence in geological models of coal resources.

Integration of this broad range of datasets can provide a very detailed and accurate model of the coal resources to ensure a secure future in mine production and operations.

REFERENCES

Vermeer, G J O, 2002. 3-D seismic survey design, Society of Exploration Geophysicists, Tulsa, 72p
THE USE OF DOWNHOLE PIEZOMETERS IMPLICATIONS FOR MODERN UNDERGROUND MINES

John Doyle¹ and Greg Poole²

ABSTRACT: An understanding of the hydrogeological environment of the strata over and about an operating mine provides some insight into the effectiveness of gas drainage, the identification of potential difficult drainage areas and the behaviour of overlying aquifers during mining.

Bore hole piezometers have been routinely installed in BHP Billiton’s exploration holes for many years. The instruments were developed as one of the outcome of NERDCC research grants completed in 1983 and 1986. Piezometers and geophones packaged to survive the difficult environments of the Southern Coalfield remain operational and are routinely read today, twenty years after their installation.

The data acquired over the years present a view of the piezometric state of the selected target seams prior to, and during mining, that would be difficult to achieve by other means.

The installation of piezometers is now a routine part of the borehole abandonment procedure. Piezometers are precisely placed against the target coal seam at depths in excess of 700 m with minimal tools and non-specialist field technicians.

The intention of this paper is to present procedures for installation of the instruments and some examples of the results as a means to encourage the greater use of borehole piezometers in the mining industry.

HISTORY

While the precise location and timing of the first vibrating wire piezometer installed by BHP in an exploration borehole in the Southern Coalfield is now lost in time, the period 1980 to 1982 is the most likely date. During and preceding this period, BHP in concert with other South Coast coal producers were involved in the enquiries relating to mining beneath stored waters which ultimately led to the creation of the Dams Safety Committee. These activities initiated an increased interest and competence in the monitoring of ground water behaviour during and after mining within the mining community.

In this same period, BHP was undertaking NERDCC funded research into the development and application of in-seam seismic for exploration. Part of this research was directed towards the bore hole to workings in seam seismic transmission technique. One of the outcomes was the manufacture of a borehole piezometer that could be economically placed in each exploration borehole during abandonment “in case” the need for an in-seam seismic survey arose in the future. At some point in this early period the incorporation of a piezometer was made to the instrument bundle. BHP and now BHP Billiton have continued the practice of routinely installing geophone/piezometer modules in the coal seam of economic interest. Although not a topic of this paper the BHPB also regularly permanently install complex geophone arrays to assist in seismic control and multiple piezometer installations to meet the requirements of the Dams Safety Committee.

THE PIEZOMETER

The vibrating wire piezometer has proven to be a reliable instrument in the Southern Coalfield. Installations made in 1982 are still functional and are routinely monitored. Piezometers have been installed to depths in excess of 800 m in HQ (96 mm) holes. Piezometers are planned in boreholes exceeding 900 m in the immediate future. Some early piezometers were constructed using a wheatstone bridge type sensor but while excellent in operation they were discontinued due a break in local supply.

A vibrating wire piezometer operates by sensing pressure on a metal diaphragm which has a taut wire stretched between the diaphragm and a stable anchor point within the instrument. To read the instrument the wire is

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“plucked” by powering an electromagnetic coil with a sweep frequency pulse. The taut wire will resonate at a set frequency related to its tension which in turn is related to the small deflection of the pressure diaphragm. This frequency signal is induced into the electromagnetic coil and then to the surface logger. The output signal is very tolerant to the effects of cable length, resistance, cross talk and electrical leakage, thus reducing to a minimum the probability of failure of installed instruments.

BHPB re-engineer commercial vibrating wire sensors into a purpose built pressure rated sonde. This sonde may also contain a number of geophone elements and possibly a digital compass. Experience has shown that commercial instruments were unreliable at depths greater than 100 m and invariably failed at depths greater than 300 m after a period of time that varied from minutes to months.

Failure appears to be due to intrusion of ground water into the instrument cable and thence to the instrument. The problem was ultimately resolved by the development of a water seal at the cable end of the sonde that prevents the movement of water along the individual conductors inside the insulation. Cables are now sheathed in polyethylene rather than PVC which markedly reduces entry of ground water into the cable. In extreme situations where high strata gas or seam gas are present then grease block cable and/or additional water seals along the cable have been utilised.

Other than small modifications as more modern materials or machining processes become available, the BHPB piezometer instrument has remained the same since the mid 1980’s

**INSTALLATION METHODS**

A number of methods have been developed to install piezometers or instrument bundles into HQ size (96 mm) exploration holes to depths of 900 m. The installation method must also ensure that the hole is totally filled with a grout that can guarantee the integrity of the future mine roof with regard to water ingress.

All of the piezometers installed prior to 2004 were installed in a sand pack with a cement grout seal above and below the installation horizon. This procedure was time expensive, generally involving two days of rig time. In 2004 a situation arose where an array of piezometers were to be installed in a shallow (160 m) NQ size (76 mm) hole. Full grouting without the use of sand packs was specified by the hydrologist.

Published papers (McKenna, 1994, Mikkelsen, 2003) promoting the grouting of vibrating wire piezometers completely within cement have been promoted by piezometer manufacturers. McKenna (1994) states “the key to the success of the grouted in installation method is that modern diaphragm-type piezometer tips require only a very small fluid volume change for pressure equalisation, and the grout can transmit this volume over the short distance from formation to the tip very quickly” Slope Indicator (2006) cite a maximum lag time of 3.5 minutes to respond to a 70 kPa pressure change through 200 mm of grout.

Following the success of the shallow hole trial the decision was made to apply this grouting method to deep exploration holes.

The success of the grout in method demands that the diaphragm of the piezometer be immersed completely in the grout without the presence of any air bubbles. The grout in method operates on the premise that minimal fluid movement is required to make the small dilation of the piezometer diaphragm. The presence of any air adjacent to the diaphragm will markedly affect the reaction time of the piezometer. The usual field solution to this problem is to remove the sintered screen from the piezometer and then to install in the inverted position ie the diaphragm facing upwards thus ensuring any air bubbles escape. The cable glanding of the BHPB unit prevents inverting the piezometer diaphragm. The coupling of the piezometer with the grout was achieved by filling the volume in front of the diaphragm with flexible epoxy filler, eliminating potential air entrapment spaces.

The elimination of the previous multistage grout/installation process with a single pass grout installation has meant that a drill rig can be released prior to geophysical logging. This equates to a saving of four to five days of rig time previously spent on stand by or grouting operations.

Piezometer installation is achieved using three people and minimal hardware.

Sufficient 25 mm poly pipe is laid out on the ground to reach the bottom of the hole.
The geophone is measured out and marks taped on the cable identifying the install depth and usually the position of the top and bottom of the seam.

A PVC ballast pipe containing nominally 20 kg of clean coarse gravel is attached to the poly pipe as a leader. The opening of the poly pipe must remain unobstructed.

All lengths are reconciled and marks made on the poly pipe at the install level of the piezometer and the collar of the hole.

The poly pipe is introduced into the hole over a large diameter wheel. It will be found that the pipe will sink in a controlled manner once the poly enters water.

When the first mark on the poly reaches the collar which indicates that the piezometer is at the correct installation depth.

**MONITORING**

Monitoring using automated loggers have captured most of the initial data from each new piezometer from the time of installation and for a period of a few days after grouting. Many of the earlier bores were not monitored after that time, particularly if they were remote from active mining. More recently a concerted effort has been made to instrument every new piezometer site and to manually read initially and ultimately to instrument every piezometer.

A logger has been selected that can be enclosed within the collar casing of the bore hole. The collar casing is extended to a height of 1.5 m above the ground and a thick walled PVC enclosure of the same diameter as the casing is attached to the top. A small solar cell (1 watt) provides long term power. The selected logger has a low power wireless data link which can be addressed within 500 m range. Experience has shown that range can vary from 50 m to 1200 m depending on site conditions. The radio link minimises the need to enter private properties and unduly impose on the residents.

**TYPICAL OUTCOMES**

**Mine 1**

A series of piezometers covering the proposed mine area of Westcliff Mine are showing draw down effects due to the proximity of the mine and the effect of in seam gas drainage. A zone of higher piezometric head adjacent to the north western extremity of the present longwall development is coincident with an area of high CO$_2$ seam gas and related poor drainage. The zone of higher piezometric head interpolated from bore hole piezometers remote from the immediate vicinity of the roadways supports the geological model and the predicted continued difficult gas drainage difficulties in that area.

**Mine 2 dyke zone/ gas**

The proposed Dendrobium longwall development is cut by a major structural zone characterised by jointing and multiple, intermittent igneous dykes. The piezometric data derived from wide spread exploration bores support the premise that the areas on either side of the dyke zone are most probably discrete features with regard to their gas properties. The north eastern region is characterised by a higher piezometric head, not all of which is attributable to gas pressure. However the piezometric pressures measured are sufficient to permit the storage of a higher volume of seam gas. Conversely the south western area is typically very low in piezometric head which also indicates that the gas saturation of the seam is also low.

These data, along with other considerations, has altered the proposed mine plan to maximise the extraction of coal to those areas initially least affected by excessive volumes of seam gas.

**CONCLUSIONS**

A method for the installation of piezometers and other instrumentation into deep exploration bore holes has been developed and successfully applied for over fifteen years. Piezometric data are routinely gathered and form part of the suite of geological information assessed in the evaluation of a mine area.
REFERENCES

Mikkelsen, P E, 2003. Piezometers in Fully Grouted Boreholes Symposium on Field Measurements in Geomechanics, FMGM, Oslo, Norway
PIKE RIVER COAL – HYDRAULIC MINE DESIGN ON NEW ZEALAND’S WEST COAST

P Whittall

ABSTRACT: Pike River Coal Limited is in the process of developing the Pike River Coal Mine in the Paparoa Ranges of New Zealand’s South Island. The mine will be developed to extract a coking coal deposit of the Brunner Seam, some 46 km north-east of Greymouth, using a combination of continuous miners and hydraulic extraction.

The mining area is located within Department of Conservation (DOC) land and is adjacent to the escarpment of the Paparoa Range and the Paparoa National Park. As such, the major restriction to planned extraction is protection of:

- The western escarpment;
- Vertical and sub-vertical rock faces;
- Permanent water courses;
- Significant vegetation; and
- Steep slopes.

The mine will use its elevation advantage to hydraulically flume coal from the working faces to the pit bottom area where it will be slurried to approximately 35% density of solids and pumps down a slurry line to the coal prep plant some 10.5 km down the Pike River Valley. The coal will be sold on the export market as a high quality coking coal for international steel making.

INTRODUCTION

The Pike River Coalfield is located on the West Coast of the South Island of New Zealand. It is located on the top of the Paparoa Range between Mount Hawera (1,189 m), and Mount Anderson (1,069 m), approximately 46 km north east of the coastal town of Greymouth.

The Pike River coal lease has been explored, contemplated, drilled, cored, sampled, planned and plotted over actively for at least the last thirty years. Access to the seam and a suitable method of extraction have prevented earlier development of the mine. In the early 1990s New Zealand Oil and Gas, a publicly listed NZ company, acquired the lease and set about the process of gaining resource consents and access agreements. The planning of the mine has been through a number of iterations in line with the submissions of the Assessment of Environmental Effects (AEE). Approval was finally gained in October 2004 to develop the mine which holds New Zealand’s largest deposit of high fluidity coking coal. Recent JORC compliant resource estimates by Golder and Associates NZ have resulted in 58 Mt of resource being calculated.

PRCL intends to develop this resource as a low ash, high fluidity, coking coal. The mine is to be developed using a combination of technologies that have been successfully implemented in New Zealand ie. road header/continuous miner development and hydraulic monitor extraction.

Following Board approval for the Project, the last quarter of 2005 saw significant work packages let to develop the road access infrastructure, the underground stone drift, the surface infrastructure, the coal preparation plant and the mine-to-export port coal transport system. The coal preparation plant (CPP) and coal transport chain from the CPP stockpiles to Port Westland (Greymouth) and thence by vessel to Port of Taranaki will be operated under contract.

With an average rainfall at site of some 6304 mm and mine location within Department of Conservation land, the construction and operation of the mine will present some significant challenges for the Pike River team. First coal

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1 Pike River Coal Limited
is expected in mid 2007 and full production ramping up over the next two years to 1.3 Mt of product coal.

This paper gives an overview of the proposed Pike River Coal Mine and also introduces some of the mine planning parameters which have effected the “final” mine plan.

LOCATION

Pike River coalfield is located on the top of the Paparoa mountain range about 46 km northeast of the port town of Greymouth on the West Coast of the South Island, New Zealand.

Fig. 1 - Location of the Pike River Coalfield

OWNERSHIP

New Zealand Oil & Gas Limited (“NZOG”) is the majority shareholder in Pike River Coal Company (PRCL) at 68.7 %, Saurashtra Fuels 10.6 % and minor shareholdings held by 31 private New Zealand and Australian shareholders. In mid 2006 Pike River Coal will undertake an Initial Public Offering and will float independent of NZOG, NZOG remaining as a major shareholder. This will make Pike River Coal (PRCL) New Zealand’s only listed coal company.

GEOLOGY

The Pike River Coalfield contains two coal measures sequences, the Eocene (Tertiary) aged Brunner coal measures and the Cretaceous aged Paparoa coal measures.

Both coal measures are exposed on the western escarpment for approximately 6 km providing a good reference point for calculating coal resources. The Brunner coal measures are the same coal measures as those at the Buller Coalfield. Typically they have only one, thick, continuous coal seam that has low ash content and low sulphur in the middle and base of the seam trending higher towards the roof of the seam.

The deeper Paparoa coal measures were formed in a lacustrine environment and as a consequence the coals have low sulphur content. At this stage, the Paparoa Seams have not been included in the current mine planning but have been estimated to contain up to 40 Mt of high grade coking coal with a lower sulphur than the targeted Brunner Seam. The Pike River coal deposit has been uplifted about 800 m above sea-level and is truncated on its western margin by a scarp and on its eastern margin by a reverse fault, the Hawera Fault. The seam dips from the
outcrop to the Hawera Fault at inclinations varying from 11 degrees to 20 degrees to the east. Small normal northeast-trending faults intersect the seam, but generally the seam thickness exceeds the throw on the faults.

The coalfield has been extensively mapped along the outcrop of the seam and on the surface of the coalfield where rocks are exposed in stream beds. A typical cross section is shown in Figure 3.

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**Fig. 2 - Overview of the Pike River Permit area**

The coalfield has been extensively mapped along the outcrop of the seam and on the surface of the coalfield where rocks are exposed in stream beds. A typical cross section is shown in Figure 3.

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**Fig. 3 - Natural scale cross-section of the Pike River Coalfield**
The main seam, the Brunner Seam, is present throughout the coalfield and varies in thickness from about 2 m to more than 13 m. The true seam thickness is generally in the range 4 m to 9 m. The coal analysis data indicates that the Brunner Seam has a typical profile of both ash and sulphur increasing from the floor to the roof. The coal is usually a hard bright coal with occasional mineral bands (carbonates), and visible pyrite usually towards the top of the seam. There are no recognisable or continuous coal plies or claystone partings within the seam. The seam has been sampled in arbitrary intervals of between 0.5 and 2.0 m.

Pike River Brunner Seam coal is a hard coking coal, with a number of favourable characteristics (i.e. very low ash and phosphorus) that give it a competitive advantage over many coking coals. The middle and base of the seam have low to very low levels of ash and sulphur in the coal, along with very high fluidity (≥45,000 ddpm). The high fluidity levels are retained for periods well in excess of one year.

MINE LAYOUT

The latest geological modelling has been carried out using Vulcan software, dividing the Brunner Seam into five horizons (although some of the horizons do not exist over the entire target area). Mine planning and scheduling has been undertaken with XPAC software. This has resulted in a powerful model that provides more accurate predictions of coal production quantity and quality.

The mine has been designed to have a minimal environmental impact through:

- Underground mining;
- Access through a stone drive (tunnel) and transport of coal in a steel slurry pipeline (Figure 4); and
- Mine planning to ensure no adverse effects on the land surface from subsidence.

The mine layout has taken into consideration several key surface features that require varying degrees of protection from mining induced subsidence. The key surface features are:

- West facing escarpment – subsidence protection for the escarpment is intended to minimise the likelihood of failure of the escarpment, which would result in toppling of the cliffs making up the escarpment. The Mining Control Zone (MCZ) for the protection of the west facing escarpment is determined by a zone extending from the crest of the escarpment or boundary of the Paparoa National Park by 20 m (horizontally) plus a distance equal to 26.5 degrees angle of draw from the top of the escarpment or boundary of the Paparoa National Park to the coal seam;
- Internal rock faces – a significant internal rock face is defined as a vertical to sub vertical face of rock of greater than 25 m in height. Three significant internal rock faces have been located within the target area. The MCZ’s for significant internal rock faces are determined by extending from the faces by 20 m plus a distance equal to 26.5 degrees angle of draw from the top of the face to the coal seam;
- Permanent watercourses – protection of watercourses is required to prevent disruption to surface drainage patterns and associated micro-environments, as well as to prevent excessive quantities of water reporting to the underground workings. The important consideration here is to prevent the formation of large cracks that could provide a hydraulic connection between the surface and the underground workings;
- Steep slopes – subsidence protection for steep slopes is intended to minimise the likelihood of slope failure. A steep slope is defined as a slope having a slope angle of more than 32.5 degrees and a length or height greater than 100 m. The MCZ’s for steep slopes are determined by a 26.5 degrees angle of draw from either side of the steep slopes to the floor of the coal seam;
- Significant vegetation – aimed at protecting areas of vegetation of significant value and/or high sensitivity to ground tilts and strains; and
- Paparoa National Park.

The current surface features have been identified from aerial photography and a digital topographic model that provides contours at 5 m interval (down to 2 m in low vegetation cover areas). Figure 5 below illustrates the surface topography.

![Perspective view of surface of the Pike River Coalfield](image)

Fig. 5 - Perspective view of surface of the Pike River Coalfield

The relationship between the width of mining panels and the depth of cover offers a means of controlling surface subsidence impacts. By controlling panel width to less than the maximum bridging width for any given overburden depth, surface subsidence can be maintained at low levels. Above maximum bridging width, subsidence is less than maximum but is difficult to control within tight limits because of natural variability. In wide panels, full subsidence can be expected.

In the particular circumstances of the Pike River Project, there is a special requirement to maintain overall stability of the overburden strata against en masse, down-slope movement. The potential for this movement is a result of the consistent seam dip toward the east. Mining will tend to cause down-slope and down-dip movement of the overburden strata. If coal is fully extracted from a down-dip area, there is potential for mass movement of the overburden strata into the mined area with potential to adversely impact on the Island Sandstone escarpment forming the western edge of the lease area.

To maintain overall panel stability, subsidence control pillars located between adjacent extraction panels are oriented up and down the seam grade, rather than across the grade. While more inconvenient from a mining perspective, an upslope-downslope alignment of the leave-in pillars will reduce the potential for overall down-slope movement of the overburden. These pillars are spaced so that the intervening panels are of the appropriate width for the intended level of surface subsidence. The pillars are intended to be of sufficient size to remain stable in the long term.
MINE LAYOUT CONTROLS

The stone drive will intersect the Brunner Seam adjacent to an existing drill hole. Pit bottom development in coal will include approximately 720 m of coal development to provide access for essential services including power, fluming transport, coal slurry holding pens and emergency water sump. Main headings will be developed from the pit bottom area to provide access to the initial mining areas to the west of pit bottom.

The mine layout has been designed to meet a range of criteria which includes:

- Integration with selected mine access;
- Resource recovery – the layout has been designed to maximise resource recovery within the available area, whilst protecting key surface features;
- Maintain flexibility within a complex structural environment;
- Optimise coal quality – over the area sulphur exhibits variability within the seam;
- Optimising the mined product and to provide a product that matches marketing specifications;
- Provision of services – layouts have been designed to adequately provide for the required services, i.e. men and materials transport, coal transport, ventilation, pumping, power, compressed air etc.

MINE DEVELOPMENT

PRCL intends to take advantage of the seam gradients within the mine and utilise hydraulic transport of coal. This therefore necessitates a number of drivage principles, including:

- Roadways will be driven to the rise at gradients that enable the efficient transport of coal by gravity. Typically, fluming grades should be greater than 4 degrees and less than 12 degrees. Optimum fluming grade (depending on flume material), is generally about 5 degrees.
- Maximum roadway gradients will also be determined by the maximum practicable grades at which development machinery can operate;
- To maximise panel recovery, entry to the panel should be at the lowest practicable point. Additionally, roadways should be driven as low in the seam as possible to maximise recovery of coal on the rise side; and
- In some cases it will not be possible to develop all areas to the rise. In these cases a suitable method of dip working will be employed.

Underground roadways will be driven to a width of 5 m and an average height of 3 m. Actual roadway shape and dimensions will vary with cutting equipment, duty, longer term requirements, ventilation, installed services etc. Development rates for continuous miners and road headers will vary with roadway grade. Roadway width may vary (ie. be narrower) in the extraction zones to increase development rates and increase extraction to roadway coal ratios.

Scheduled coal production commences at pit bottom in mid 2007, utilising a single roadheader development unit. Once pit bottom drivage has been completed and the shaft/fan installation complete, a second development unit is to operate in tandem with the first to drive main headings. A roadway will be driven in a north westerly direction from pit bottom to hole out at the surface at an area of low depth of cover as soon as possible. This is to provide an additional intake airway. At this point a third development unit will be added to carry out additional development drivage.

EXTRACTION

Hydraulic monitor extraction has been proven to be the most suited to complex mining conditions on the West Coast of New Zealand and has been selected for the Pike River conditions. The method offers the best recovery in thick and steady dipping seams. It also provides the highest productivity, greatest flexibility and lowest operating cost. As the coal seam is located at a high elevation, almost all coal can be transported using gravity.

Operation of the monitor extraction system will be on the basis of three available units to provide one operating unit, one unit that will be in the process of dismantling and retreat, and a third standby unit that will also be available for production as required. This will provide scheduling flexibility to allow blending to occur to ameliorate the sulphur variability.
During hydraulic extraction, it is planned to mine the lower portion of the seam first. This will result in the selective recovery of the better quality coal. A separate working section within the higher plies will then be extracted.

Hydro monitor extraction is scheduled to commence in mid to late 2008, some 16 months after commencement of development at pit bottom. Components in the mine’s systems have been designed with a production capacity of up to 1.4 Mtpa, which will include 0.2 to 0.3 Mtpa from roadway development and 0.8 to 1.1 Mtpa from the hydro monitor unit. Planned peak production is up to 1.3 Mtpa.

Mined coal will be flumed to the pit bottom area prior to which some stone dilution will have been removed by stone traps. At pit bottom the coal will be crushed to <35 mm before the coal/water slurry is conditioned to 25 % pulp density (i.e. 25 % coal, 75 % water). However the system is designed to accommodate peak flows of 45 % pulp density. The slurry will be transported through a 275 mm inside diameter steel pipeline about 10.6 km downhill to a coal preparation plant. At the coal preparation plant the coal will be washed, then separated from the water, screened and stockpiled in two grades according to its quality awaiting dispatch.

The following mine plan shows detailed mine planning in the body of the resource where the Measured and Indicated resources are located. The south eastern area contains approximately 4 Mt of coal which has been designated inferred status due to borehole spacing. A further 15 Mt is located in the north of the resource under the Paparoa National Park. The National Park was formed after Pike was granted the mining lease and contains provision for the coal to be mined following successful demonstration of minor surface impacts.

**GAS AND VENTILATION**

The coal seam has a medium to high gas content throughout the resource area. Methane is the dominant gas at >90 %. The trend in content is variable, however a number of earlier samples have been found to be incorrectly tested. Typical of seam gas contents is a low Q₁ value (< 0.5 m³/t).

Gas contents are generally low throughout the seam with total in-situ contents of 3 to 6 m³ generally. Recent sampling has determined a seam gas content of 7.0-7.5 m³/t at the proposed seam entry location. This is at a depth of 85 m. This gas content is considered to be difficult to control by ventilation means alone and in seam gas capture (pre-drainage) will be used as part of the roadway development process. PRCL will aim to reduce seam gas to < 3 m³/t prior to mining, however where insufficient lead-time is possible, a maximum content of 6.5 m³/t will be sought so as not to pollute the intake airways with rib emissions. In thick seam mining a more significant impact is content per square metre as the whole seam is removed during hydraulic extraction and the gas is liberated to the return airways.

Ventilation quantities have been determined for each of the mining sections with minimum air quantities within the roadways to dilute the rib gas emissions assuming a maximum 6.5 m³/t gas content and moderate seam permeability. The seam also displays variable permeability in region of the Hawera Fault. Similarly the content increases away from the escarpment and with depth. Ongoing assessment is in progress to determine the potential rib side emissions based on reservoir modelling and recent results of adsorption testing. The main fans will be operating within an envelope between 1 kPa to 1.6 kPa at air quantities from approximately 135 m³/s to 240 m³/s. The fans for this duty range will be axial flow fans.

A real time monitoring system will be established for all gas and ventilation parameters underground. The system will report to the mine’s control room and will also be web interfaced to allow interrogation by mine officials remotely via the internet. This will display all details of gas monitoring, underground ventilation and main fans, as well as the mine’s pumping system.

**HUMAN RESOURCES**

It is intended to operate the mine three shifts per day and seven days per week. The peak employment level will be approximately 140 operations and support personnel. The maximum employment level will provide for three development units and a hydro monitor unit, underground services and surface facilities.
Developing the Pike River Coal Mine (PRCM) in the current coal industry cycle will also present PRCL with similar challenges as those faced not only by New Zealand’s Solid Energy, but by all mining companies in Australasia. New Zealand and particularly the West Coast have a very tight labour market and Australian coal mines, offering substantial rates of pay, are also finding demand for skilled trades and operators is not being met by local supply. PRCL have recruited a Human Resources Manager and a HR Support 18 months prior to the commencement of coal operations to address issues including skills requirements, recruitment, housing and infrastructure needs, remuneration packaging, competency development, establishing relationships with other coal producers and education/training providers for combined development of training programmes. Excellent initiatives on the West Coast such as The Digger School by the Tai Poutini Polytechnic will be supported and similar initiatives for underground workers explored.

Fig. 6 - Mine layout
CONCLUSIONS

Pike River Coal will have the advantages and challenges of any new Greenfield venture. The advantages being that Pike will not have the shackles of past operational and management errors at the mine; it has a clean sheet to plan, design, construct and operate the mine with state-of-art technical and management systems; the opportunity to recruit and develop a workforce with aligned goals for the project’s success; and to achieve benchmark environmental standards for an underground operation. The challenges for Pike will be the effective application of the hydraulic mining technique to achieve consistently high output; recruiting and retaining the above mentioned workforce; and the management of the mine’s effect on the surface features and general environment from its operations.
AN INVESTIGATION INTO UNDERGROUND MINE INTERACTION WITH OVERLYING AQUIFERS, HUNTLY EAST MINE, NEW ZEALAND

Glen Guy¹, Winton Gale², Brett Sinclair³, Dean Fergusson¹ and Bill Farnworth¹

ABSTRACT: In recent years, Huntly East Mine has operated at a depth range of approximately 100 m to 220 m below a Quaternary aged clay, sand and silt aquifer that is connected to a nearby large river system (Waikato River). A key issue for mine planning and environmental management has been the development of mine design criteria to allow efficient mining of the reserves and to maintain the integrity of the aquifer. A case study and back analysis at Huntly East Mine is presented, which investigates the overburden conductivity and the impacts caused by mining-induced caving. The case study includes:

i. computer modelling of the mine geometry, caving and overburden fracture networks created;
ii. field investigation to develop an engineering geological model of the overburden within the goaf to validate the goaf geometry as defined by the computer generated model;
iii. in situ field measurement of overburden conductivity in the pre- and post-mining condition;
iv. interference testing across the goaf to determine the level of interconnectivity; and
v. measured water pressure profiles above the mine.

INTRODUCTION

Pillar extraction mining within the Renown and Kupakupa Seams of the Waikato Coal Measures is undertaken at Huntly East Mine, New Zealand. The seam is typically 20 m thick, and extraction is undertaken by double lift and pillar pocketing methods. The location and typical panel layout is presented in Figure 1. An indicative stratigraphic cross section from East Mine is presented in Figure 2.

The mining methods and layout adopted have been developed on the basis of maximising extraction while restricting subsidence and, importantly, maintaining the integrity of the Tauranga Group aquifer which is composed of unconsolidated Quaternary aged sediments.

The Waikato River flows over the mining area and is hydraulically connected to the Tauranga Group aquifer. The depth of mining is variable however recent mining has been within a depth range of approximately 140-230 m. The thickness of the overlying Tauranga Group sediments ranges from approximately 20-40 m. Subsidence over the study area mine panels ranges from approximately 1-1.2 m.

A detailed investigation program was undertaken for panel N51 to study the potential groundwater impacts caused by mining and to obtain information to optimise mining operations. The program included:

i. surface subsidence monitoring,
ii. develop an engineering geological model above the goaf to evaluate the mining induced fracture geometry using post mining drilling above the panel,
iii. monitoring of strata caving using a surface to seam extensometer,
iv. monitoring of water pressure drawdown within the overburden using pre-placed and post emplaced piezometers,
v. packer testing to measure ground conductivity above the goaf,
vi. injection tests to assess fracture connectivity.

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Fig 1: Location and mine layout at East Mine, New Zealand.

Fig. 2: General stratigraphic section and test borehole locations.
Computer modelling of the caving process, fracture distributions and the hydraulic conductivity above the extraction panel was undertaken prior to the monitoring programs. The results of both the modelling and the investigation and monitoring program have been utilised to assess the mining impact on overburden conductivity. This information also forms part of an ongoing planning process to evaluate the impact of other mining techniques on groundwater flow patterns within the overburden. The results of the field study and the application of computation modelling to predict overburden and conductivity is presented.

**OVERBURDEN CHARACTERISTICS AND PROPERTIES**

The overburden is composed typically of very weak to moderately strong mudstones and clay rich interbedded fine sandstone and siltstone. Sandstone units exist in the sequence however their properties are variable. The overburden has been characterised on the basis of geophysical logging and laboratory testing of core. An indicative strength profile through the overburden is presented in Figure 3 on the basis of unconfined compressive strength of each unit. The Tauranga Group is approximately 20 m thick above N51.

**IN SITU HYDRAULIC CONDUCTIVITY MEASUREMENTS**

Hydraulic conductivity results from packer tests performed prior to mining are plotted in Figure 4 relative to depth. Some of the data was classified on the basis of formation without any depth and these are presented at the top of the plot. The packer interval varied for this data set and as such the results should be seen as indicating the sensitivity sampling a range of possible conductivity within the overburden. The silty sandstone units at Huntly typically have a conductivity of $10^{-6}$ to $10^{-7}$ m/s. The conductivity of the finer grained rock matrix is considered to be in the range of $10^{-10}$ m/s to $10^{-11}$ m/s, however the rock mass fabric can result in considerable local variation. Undisturbed rock fabric data for the N51 overburden is not available and the values are based on tests performed in nearby drillholes.

Variation from the $10^{-10}$ to $10^{-11}$ m/s hydraulic conductivity range in the fine grained strata is typically caused by dilated bedding planes and joint planes in the overburden allowing water movement in the packer zone. Values in the $10^{-6}$ to $10^{-7}$ m/s range are inferred to represent structured ground. The data set suggests that the field conductivity of the overburden within the Huntly Coalfield is in the range of $10^{-6}$ to $10^{-10}$ m/s. Based on the assumption that there was no bypass around the packers in the tests, the variation within the data set suggests that the packer tests have been performed on a range of fractured and unfractured drillhole intervals in the test programs. This is supported by the results from short test intervals returning a greater range of results than tests performed on longer drillhole sections. It is possible that some fractures are “healed” in proximity to muddy units at the base of the Tauranga Group, however for the purpose of this study the data is taken as indicative of the range of background conductivity.
PRE-INVESTIGATION COMPUTER MODEL

A computer model of the overburden and mine extraction of N51 panel was developed using FLAC 2 dimensional code. Development of this model preceded the field investigation program. The model was undertaken to improve the understanding of caving related fractures and overburden behaviour above the extraction panels. The model allowed an estimation of changes to the overburden hydraulic conductivity induced by caving and subsidence movement. The resultant mining induced fracture geometry generated from the computer model is presented in Figure 5. The induced fracture geometry indicates a combination of:

i. bedding plane shear,
ii. shear fracture of strata,
iii. tension fracture and potential reactivation of pre-existing joint planes.

The planned location of the drillholes for the post-mining hydrological study is presented on Figure 5. It was anticipated that the drillholes would intersect a range of sub-vertical and horizontal mining induced fracture systems.

ESTIMATION OF MINING INDUCED HYDRAULIC CONDUCTIVITY

The mining induced changes in conductivity have been assessed on the basis of the fracture apertures and fracture distribution calculated from the ground displacements within the model. The approach used is to calculate the dilation of the strata subsequent to fracture formation and relate this to the aperture of the fracture within which water may flow. This has been done for:

i. vertical and sub vertical fractures to assess the vertical conductivity, and
ii. bedding planes to assess horizontal conductivity.

The equivalent material conductivity has been calculated from aperture flow within a fracture. The conductivity (k m/s) is estimated from the flow quantity through a 1 m$^2$ area with unit pressure gradient. This then simplifies to solve k as approximately equal to:

\[ k = t^3 \times 10^{-6} \text{ m/s} \]

Where; “t” is the hydraulic aperture (m).

It has been assumed that there is one fracture per element in the model and that the aperture is equal to the average dilation less 0.5 mm. This should be considered to be an estimate, and the data has been analysed on the basis of relative impacts.
The model can be interrogated to determine the average vertical conductivity for each one meter layer across the model above the extraction zone. The average conductivity for each succeeding layer is calculated and plotted to give a vertical conductivity profile. This profile does not specifically simulate groundwater flow pathways. It assumes that flow can occur along bedding planes to allow the vertical pathways to be activated with relative ease. The extensive mining induced shearing of bedding planes in the overburden would provide evidence for this situation.

The average vertical conductivity profile over the ribline area is presented in Figure 6 for the N51 modelled geometry. The data is plotted on the basis of a running 5 m vertical section of equivalent conductivity. The results indicate that groundwater flow downward toward the extraction zone would need to occur via a network of vertical and horizontal planes as opposed to any single connection plane.

There appear to be three zones formed.

1. The caved zone where there is direct open flow.
2. A highly fractured intermediate zone linking the caving zone to the overburden above (tortuous zone). This zone is characterised by extensive shear fracturing over the abutment areas.
3. Tortuous flow zone, which has layers of high conductivity separated by zones of low conductivity. Flow in this zone requires an interconnected network of vertical and horizontal fractures to form. Flow would be tortuous and this section forms the effective flow control zone between the intermediate zone and the surface aquifer.

FIELD INVESTIGATION

Five drillholes were programmed to be drilled perpendicular to the goaf edge (Figures 1 and 2), inclined and orientated to intercept the anticipated fracture geometry as indicated from the computer model. Two of the drillholes were positioned within the central part of the underlying mined panel, inclined (55° and 75°) and fully cored and subsequently geotechnically logged to ascertain visual confirmation of the rock mass quality and extent of the fracture geometry. RQDs, fracture intensity and defect orientation were recorded and used to develop a 2-dimensional Engineering Geological model across the goaf zone. This is presented in Figure 2.

Terminal packer tests were undertaken at discrete intervals within the two central drillholes during the drilling process to assess the strata conductivity. Piezometers were subsequently installed within one of the central drillholes and three subsequent instrumentation holes drilled adjacent along the same strike. Piezometers were installed at regular intervals within the overburden to assess:

i. pore pressure changes indicative of fracture connectivity,
ii. the water levels in the ground, and
iii. subsequent long term variation in water levels.

POST MINING WATER PRESSURE AND STRATA CONDUCTIVITY MEASUREMENTS

The results of the N51 terminal packer tests and the regional in situ data set were compared and are presented in Figure 7. The consistently high hydraulic conductivity values from N51 clearly indicate the impact of mining throughout the overburden where tested. The data indicates mining induced conductivity in the range of $10^{-7}$ to in excess of $10^{-6}$ m/s. The results are interpreted as representing a combination of flow through open bedding planes, mining induced fractures and reworked joint planes. It was not possible to discriminate the vertical and horizontal components of the flow system.

Following the completion of drilling and installation and development of the piezometer network a series of injection tests were carried out using a straddle packer set-up within the central drillhole. Real-time hydraulic head monitors were installed into the remaining drillholes and their response recorded at different injection pressures. Results from injection tests indicated that there was no direct connectivity of the fracture system between piezometer screens positioned at different elevations within the overburden.
The piezometers above the goaf provided data on the water pressure distribution from the Tauranga Group to the mine. The results are presented in Figure 8 on the basis of piezometric level (water table level) relative to depth. The results indicate that total head loss occurs in the caved zone, and partial loss extends upward toward the base of the Tauranga formation. These results indicate a significant impact of the mine on the water pressure distribution within the overburden and provide information with which to undertake computer model evaluation.

WATER INFLOW ESTIMATES

The amount of water estimated to result from vertical flow through the goaf in this panel is in the range of 50-120 \( \text{m}^3/\text{day} \).
BACK ANALYSIS OF RESULTS

Back analysis was undertaken to assess the results of the field investigation and computer model in terms of matching the water pressure profile, field conductivity and water inflow estimates into the panel.

PACKER TEST RESULTS

The packer test data indicates conductivity in the range of $10^{-6}$ to $10^{-7}$ m/s. If one assumes this to be an estimate of the vertical conductivity then the inflow would be at least two orders of magnitude too high. Therefore direct use of the packer data above is not appropriate for assessing the vertical conductivity and inflow characteristics of the overburden. The interference testing result is consistent with this interpretation.

COMPUTER MODEL FLOW MATCHING

In order to obtain a better estimation of the overburden vertical conductivity, a back analysis was undertaken using computer modelling to match flow and the water pressure profile obtained from the field study. The model developed for this was a flow model which had a range of conductivity layers above the goaf zone, principally derived from the previous N51 model, and a range of \textit{in situ} conductivity profiles, principally derived from the background testing data. The conductivity distributions above the goaf and for the in situ ground surrounding the panel were varied from the N51 base case to assess the impact that other combinations may have. The aim of this study was to undertake a reality check on the N51 model results and the conductivity estimation process. The use of a simplified flow model allowed a range of options to be evaluated. The model is presented in Figure 9. The assumptions in the model are that the Tauranga Group silt and sand had a uniform conductivity of $5 \times 10^{-5}$ m/s and the horizontal flow at the boundaries was constant. The work program was conducted in two stages. The first stage was to evaluate the most likely \textit{in situ} conductivity profile on the basis of a match to the modelled field data. The second stage was to assess a range of possible situations which need not match the modelled results.

The options assessed were:

1. Most likely upperbound conductivity obtained from the N51 model.
2. Most likely lowerbound conductivity obtained from the N51 model.
3. Assumed no significant conductivity impacts in overburden – overburden in the range of $10^{-7}$ to $10^{-8}$ m/s.
4. Assumed no significant conductivity impacts in overburden – overburden in the range of $10^{-8}$ to $10^{-9}$ m/s.
5. Assumed high fracture density in goaf with overburden conductivity approximately $10^{-6}$ m/s.
6. Best estimate upperbound with two aquitards in the overburden.

The inflow rate for these options is presented in Figure 10 together with an estimate of the likely inflow from initial calculations. The estimate is considered to be “ball park” and for the model result to be close to the estimate is a reasonable result. The inflow rate was calculated as the inflow rate from the model multiplied by the surface area of the panel. The surface area used was 117 000 m$^2$. 

![Diagram](image-url)  

\textbf{Fig 9: Simplified model for flow and water table matching.}
The range of conductivity profiles modelled as close to the initial N51 data (adjusted for the increased depth) is presented in Figure 11a and 11b. These profiles cover the range of in situ strata adjacent to the extraction panels and that above the extraction panels. The water surface level obtained through the centre of the goaf is presented in Figure 12. The measured water levels are presented for reference. The model results indicate a good match.

The data presented is based on the level of the water table (below ground) above the goaf. An in situ condition would be a vertical line at the origin which indicates that the water table is at the surface all the way down the section. As mining occurs out flow into the mine exceeds inflow from the Tauranga Group and reduces the water head at that location. The results indicate that drawdown is occurring virtually to the base of the Tauranga formation. This is confirmed by the measured piezometric gradient from N51 (Figure 8).
It appears that the shape of the drawdown curve is similar for the modelled results and the inflow rate is within an acceptable range for this case. Minor variation in the conductivity in the initial 60 m of the model or variation in the Tauranga silts could provide a more refined match, however the key issue is whether the modelled result provides a reasonable match in the first instance. This appears to be the case, and indicates that the conductivity distribution developed in the N51 model provides a good correlation to the actual measurements.

**REVIEW OF OTHER CONDUCTIVITY POSSIBILITIES**

During the course of this study a number of other conductivity options were assessed. The options were based on combination of measured information and various scenarios which were of interest to assess. The options assessed were:

i. Virgin ground surrounding the panel and a high conductivity within the overburden above the goaf. This was based on packer measurement over the N51 goaf which indicated conductivities of $10^{-5}$ to $10^{-7}$ m/s. This value is within the range anticipated for certain sections, but not for the total section. It is likely that the packer data reflects the total horizontal and vertical conductivity and as such cannot distinguish the vertical conductivity component required for this analysis. The results of this option are presented in Figure 13 for the water table section and in Figure 10 for the inflow values. It is clear that this does not provide a good match in terms of water head profile shape, nor inflow potential.

ii. Virgin ground throughout the model except in the caved zone. This model is based on the assumption that there is no significant vertical connectivity in the strata above the caved zone. There were two options modelled, where the strata conductivity varied from $10^{-8}$ to $10^{-7}$ m/s and one where the conductivity varied from $10^{-9}$ to $10^{-8}$ m/s. The results are presented in Figure 13 and the inflow information is presented in Figure 10. It is clear that the shape of the curves is not correct, although the inflow has been good fit for the lower conductivity profile.

There are many other combinations of conductivity that could be assessed and it is anticipated that various combinations would be able to match the data. Various combinations of vertical and horizontal conductivity within the virgin strata may vary the results. However, this study demonstrates that the results from the earlier N51 caving model provide a credible match in the first instance. The assumption of no significant vertical conductivity other than virgin conditions was not seen as credible. Similarly, the assumption of major connection ($10^{-5}$ to $10^{-6}$ m/s) was not credible either.
The modelled profile is however a combination of these values where there are sections which display high conductivity and sections of low conductivity. The mix of such layering appears to provide the best estimate for the range of options assessed. The results indicate that the mix of vertical conductivity within the tortuous zone will significantly influence the profile and the inflow potential.

EFFECT OF AQUITARDS IN THE OVERBURDEN

For the purpose of this study an aquitard is considered to be a unit which has a significantly lower conductivity than the surrounding materials which can influence the flow system. An aquiclude is an aquitard with conductivity similar to clay or clay rich rock material (i.e. less than $10^{-10}$ m/s).

On the basis of the background test data, it is possible that clay rich materials exist at least locally under the Tauranga Group. The effect of two aquitards each of 2 m thickness with a conductivity of $5\times10^{-10}$ m/s located at (50 m and 70 m) was assessed to determine the water table characteristics which would result from such units should they exist. The results are presented in terms of water table level in Figure 14. It is clear that where there are significant aquitards, the water table drops dramatically immediately below the unit. Recharge within the goaf zone has to be via horizontal flow, rather than vertical flow from the Tauranga formation. The upper aquitard has the ability to hold the full water head in both instances, and the water pressure below is dependent on lateral inflow and the minor flow through the aquitard.

In general, the profile characteristic of the models with aquitards does not fit the data from N51. This indicates that the overburden rocks, whether clay rich or not, have a significant fracture fabric which allows flow at rates greater than intact clay units. This does not preclude the existence of clay layers within the Tauranga formation which may exist locally and isolate flow from the Tauranga formation.

CONCLUSIONS

The investigation program has provided characterisation of the impact of mining on the overburden. Field data, together with computer modelling, has been used to provide an understanding of the fracture geometry and hydraulic properties above the goaf.

The estimate of vertical conductivity above the goaf of N51 provided by the computer model displays a good correlation with the measured head profile, and provides realistic inflow values.

There are many other combinations of conductivity that could be assessed and it is anticipated that various combinations would be able to match the data. However, the results of this study demonstrate that the computer model developed provided a credible match in the first instance.
The assumption of no significant vertical conductivity induced above the goaf was not seen as credible. Similarly, the assumption of major connection above the extracted zone ($10^{-5}$ to $10^{-6}$ m/s) was not credible either.

The modelled profile is however a combination of these values where there are sections which display high conductivity and sections of low conductivity. The mix of such layering appears to provide the best estimate for the range of options assessed.

The data suggests that there are no major aquitards within the overburden above the extracted panels, despite the high clay content of the sequence. This indicates that the overburden rocks, whether clay rich or not, have a significant joint fabric which allows flow at rates greater than intact clay units. This does not preclude the existence of clay layers within the Tauranga formation which may exist locally and isolate flow from the Tauranga formation.

The results of this program have been incorporated into the ongoing assessment of other mining options. The use of field measurement to assess fracture geometry, water pressure profiles and overburden caving is seen as an essential part of the evaluation process.
WATER INFLOW ISSUES ABOVE LONGWALL PANELS

Winton Gale¹

ABSTRACT: The aim of this paper is to discuss the issues which relate to surface water inflow through the fractured overburden above longwall panels. The information used is a combination of field experience and computer modeling. Computer models used in this study simulate the fracture process in the geological units throughout the overburden. Analysis of the mining induced fracture patterns and in situ joint patterns allows an estimation of the hydraulic conductivity within the overburden. The cubic flow relationship has been used in examples presented.

INTRODUCTION

The occurrence of water inflow into longwall panels has been recognised and studied over a long period of time. Records of the inflow events and water make into mines allows a review and comparison methods of prediction.

Much of the published recorded information of water inflow has come from judicial inquiries and previous ACARP studies. Particularly useful information has been obtained from studies conducted in the Bowen Basin (Klenowski, 2000). Recent developments in computer modelling have allowed simulation of rock fracture, caving and stress redistribution about longwall panels with increasing confidence (Gale, Mark and Chen, 2004; Gale, 2004 and Gale, 1998). The models are being assessed against field monitoring and have significantly increased the understanding of caving related fractures and resultant hydraulic conductivity within the overburden.

The aim of this paper is to present an overview of the factors which relate to water inflow and those which may mitigate the impact of the fracture zones, which are created by longwall extraction. Published data and insights obtained from computer models are discussed.

FIELD EXPERIENCE OF WATER INFLOW

Experience of water inflow into mines has been reviewed in terms of the relationship between panel geometries and associated water inflow. The data collated is associated with ACARP Project (C13011).

The data is categorised in terms of confirmed inflow and no flow. The sites having flow may be based on actual inflow experience or piezometer monitoring data of the site which indicates the height at which inflow would occur. Sub-sets of the data have been collated for sites at which remedial repair has been conducted to provide a water resistant seal against future inflow. The term connection implies that surface water can enter the mine. It does not imply an inundation or direct connection via a single fracture.

The results of the study are summarised in Figure 1, for which the data is presented relative to depth and panel width. The results show that for situations of normal rock head, without significant aquacludes, panels with a width to depth ratio greater than one typically show confirmed connection. One site shows connection with a width to depth ratio of approximately 0.75. Panels with a width to depth ratio of less than 0.4 show no connection. The sites categorised as having aquacludes are those which have significant clay layers (i.e. 4 m+) between the aquifer and the normal rock section or have low permeability aqueous deposited silt layers above the normal rock section.

Examples of sites for which the existence of significant clay layers have controlled the inflow of water for geometries having width to depth ratios greater than one include Crinum Mine and is inferred for Gordonstone (Kestrel) Mine. These sites have significant sections of clay rich layers which are sufficiently compliant and have low permeability to maintain an effective seal between the water source and the fracture zone. These are relatively unique in the overall database.

¹ SCT Operations Pty Ltd
Similarly, sites at Oaky Creek for which inflow was noted were subsequently repaired by ripping and compacting the surface to perform as a low permeability membrane (generally 1e-8 m/s or less) above the fractured rock section.

The sites at Crinum, Gordonstone and Oaky Creek (repaired surface) are consistent with the concept of having a low permeability section which is able to control the flow between the water source and the mining induced fracture zone.

There are instances of ash dams for which limited flow occurs or which remain intact above extraction panels. The permeability of subaqueous deposits within ash dams are typically low (1e-9 m/s or less) whereas the permeability of the overburden below the dam may be several orders of magnitude higher. Mannering Creek ash dam is one instance which was undermined by Wyee State Mine (Longwall 4) without major issue. It is not conclusive (to the author) as to whether it was breached and subsequently healed. This instance is viewed as an extension of the aquaclude scenario for which the compliance of the low permeability silt/clay lining is sufficient to survive the strains imposed by mining, or can subsequently “heal” the initial cracking. Mining under Lake Macquarie (Wyee longwall panels) has occurred without significant impact, however these occurrences are considered to fall within the aquaclude category rather than be part of the “normal rock section” data set. It is considered that the base of the lake is composed fine unconsolidated silts which have very low permeability characteristics.

Mining has occurred under the Pacific Ocean at Burwood Mine, however the mine geometry had a width to depth ratio less than 0.3.

Overall, it appears that for a “normal rock section” (without aquaclues), longwall panels with a width to depth ratio above one have a high probability of connection and inflow. Panels with a width to depth ratio less than approximately 0.4 have not exhibited any connection. Panels of width to depth of approximately 0.75 have exhibited connection. Unfortunately the data set is not sufficient to define the transition point in more detail.

**COMPUTER MODELLING INSIGHTS**

Computer modelling of overburden caving and its comparison to typical subsidence characteristics has been undertaken as part of the project. The results of this, in relation to the height of cracking and induced vertical conductivity, have been compared to the results presented above. This has involved taking a number of Hunter Valley geological sections and modelling the effect of different size longwall panels. In this way the height of cracking and the subsidence created can be compared to the regional subsidence experience and the potential for

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**Fig 1:** Water inflow experience with longwall panels.
inflow at different width of panel to depth of mining can be compared to the results presented in Figure 1. The depth of mining in the modelled data set ranged from 150 to 300 m.

The stress field is based on regional estimates and the overburden conductivity is based on regional data (Gale, 2004).

The model used is based on FLAC 4 code, and has coupled mechanical and fluid interaction, such that the water pressure and flow is modelled together with mechanical ground movements.

The subsidence obtained from the models is presented in Figure 2 together with the regional subsidence information of the published Newcastle and Western Coalfield. This data is presented in terms of percent maximum subsidence relative to width/depth of the panel. The modelled results are consistent with the regional data and confirm the models’ ability to:

i. simulate the overburden deformation characteristics in a suitable manner; and
ii. simulate the goaf loading and compaction characteristics.

![Fig 2: Regional subsidence data for Hunter and Western Coalfields. NOTE: Local data and modelled data included.](image)

**OVERBURDEN CRACKING RELATIVE TO PANEL WIDTH**

The mode and extent of cracking within the overburden for panels of 0.5, 0.75 and 1 x depth are presented in Figure 3. This plot shows the overburden section together with the fracture distribution relating to each panel width. The results show that cracking connects to the surface for panels of 1.0 x depth. The 0.5 depth panel shows no connection and the 0.75 depth panel shows no connection but does display additional cracking between the main cracking zone and the surface. The 0.75 depth panel is considered to represent a transitional case.

The height of cracking relative to panel width is presented in Figure 4 and indicates that the height of cracking is the range of 1-1.2 times the panel width. Experience at other sites indicates that this range may extend depending on the geological characteristics.

**INTERPRETATION IN RELATION TO INFLOW**

The fracture networks created in the overburden will create a conductive network within the “Permian” rock (fractured rock above the extraction panel). However, it is important to assess the effect of recent sediments or soils on the surface when assessing the overall impact of the fracture networks created in terms of the inflow characteristics from surface or near surface sources.
Fig 3: Height and mode of rock fracture for various panel widths.
If soft low conductivity clay like materials exist then they may act as aquacludes or else significantly restrict the inflow rate from water sources. Experience to date indicates that the material properties of aquicludes would need to be similar or softer than wet clay. Example of this could relate to situations where subsidence fractures have been remediated by the placement of clay or silty clay layers. Aqueous silt layers may also provide a similar function under ash dams and lakes. The existence of unfractured clay beneath water bearing basalt units at Crinum would also fall into this category. Alternatively, a 10 mm crack through the surface cover can allow significant inflow. Therefore, in order to assess the inflow rates, characterisation of the surface materials and their conductivity is required. The impact of mining induced strain and resultant change in conductivity of those materials is required to assess the way in which water is transmitted into the “Permian” fracture system.

The impact on the integrity of surface aquifers with also require an assessment of the flow and recharge capability of the aquifer in relation to the flow capacity of the “Permian” fracture system above the extraction panel.

REFERENCES


DETERMINING THE CONTROLS FOR STRATA GAS AND OIL DISTRIBUTION WITHIN SANDSTONE RESERVOIRS OVERLYING THE BULLI SEAM

ABSTRACT: The continuing and effective management of gas within the sandstones overlying the Bulli seam mines of BHP Billiton Illawarra Coal is required to ensure safe and productive mining operations. Recent surface exploration has also detected the presence of oil accumulations in these sandstones which have the potential to impact on future mining operations.

Some of these hydrocarbons are located within the longwall relaxation zone of the overlying strata and, as a result, can migrate to the goaf and active workings subsequent to extraction.

A number of new exploration techniques, which are in common use by the petroleum industry, have been adopted by Illawarra Coal in order to more accurately locate these zones and determine the potential impact on future mining.

These techniques include:

- Advanced analysis of downhole geophysics to determine the location and extent of strata gas horizons.
- Specialist interpretation of 2D and 3D seismic to detect gas zones.
- Modeling of multiple data-sets to determine controls on gas distribution and composition.
- Detailed geological and chemical analysis of the oil-bearing horizons to gain a better understanding of the petroleum system and the controls to its distribution.
- Geotechnical studies of the overlying strata to determine the extent and nature of post-mining strata relaxation in comparison with the location of the hydrocarbon zones.

The results of these studies have formed the scientific basis for the development of more effective technologies to manage the impact and potential impact of strata oil and gas on the underground extraction of coal.

INTRODUCTION

BHP Billiton Illawarra Coal operates four underground coal mines in the Wollongong region of the Southern Coalfield of the Sydney Basin (refer to Figure 1). These mines extract coal under varying depths of cover with the deepest being the three Bulli seam mines – Appin Douglas and West Cliff.

The management of gas in these mines has been a concern ever since the commencement of mining in the Coalfield in the 1800s. Gas is produced not only from the mined seam but also from the underlying seams and the overlying strata as a result of strata relaxation during longwall extraction. Control of gas emissions into the workings is achieved through a number of technologies including ventilation, in-seam pre-drainage, cross-measure post-drainage, and surface goaf and strata drainage.

The identification of gas reservoirs prior to mining provides an important tool to enable effective gas control technologies to be used during mining operations. Assessment of seam gas reservoirs is a standard component of surface and underground exploration. The identification of strata gas reservoirs prior to mining requires special techniques that were originally developed for the petroleum industry.

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In a similar fashion, specialist techniques are required for the collection and analysis of strata oil. Assessment of these liquid hydrocarbons is required to ensure that, as a result of the ingress of any liquids into the goaf or workings, there are no adverse impacts on mining operations or personnel.

The mines are overlain by an interbedded sequence of sandstones, mudstones, claystones, and shales of the Triassic Narrabeen Group, Hawkesbury Sandstone and the Wianamatta Group. The principal reservoirs for strata oil and gas are the Coal Cliff, Scarborough and Bulgo Sandstone Formation (refer to Figure 2).
The techniques of geophysical logging and log analysis are mainly applied in the petroleum industry for geological purposes and reservoir evaluation. The variant of this used in mining applications is known as slimline logging and involves a less comprehensive suite of tools. The tools typically run by BHP Billiton are:

- natural gamma ray log,
- density log,
- sonic log,
- cement bond (full waveform sonic) log
- neutron porosity log,
- caliper log,
- resistivity log (laterolog),
- temperature log,
- borehole survey log,
- acoustic scanner.

From the density log it is possible to determine basic lithological information on the coal seams and their ash content; the sonic log gives information on rock strength; stress directions and fracturing information are derived from the acoustic scanner; porosities come from the density and neutron logs; and shaliness (clay content) is derived from the gamma and neutron/density logs. Descriptions on how these parameters can be derived is found in textbooks such as Rider (1995) and Hearst et al (2000). Hatherly et al (2004) also developed the methodology for use in the coal fields of the Sydney and Bowen Basins. It is this approach that has been utilised in the present work.
The presence of gas within rock pores can also be inferred from the porosities derived from density and neutron logs. This is a straightforward procedure but it is qualitative in nature.

From the density log, rock porosity is determined by comparing the measured density with an assumed density for the solid components of the rock. The difference between these is attributed to the amount of pore space present which is assumed to be water saturated. It therefore follows that if the pores contain gas as well as water, then the observed density will be lower than for the water saturated case and the inferred porosity is larger.

In the case of neutron logging, this log mainly responds to the amount of hydrogen present in the rock formation. In clastic rocks, hydrogen is mostly present in water – either in the free water within the rock pores or within the bound water associated with clay minerals. Because of the presence of bound water, a neutron log calibrated to give an accurate porosity in a clean quartz sandstone will usually suggest higher than actual porosities.

In clastic rocks, the porosity indicated by neutron logging is therefore typically greater than the porosity derived from density logging. However, when gas is present in the rock pores, the neutron log will not be responding to as much hydrogen as it would if the pores were fully water saturated. The indicated porosity is therefore lower – typically lower than the porosity from the density log, especially if the density log is suggesting higher porosities due to the reduced water saturation.

When the neutron porosity is lower than the density porosity, a porosity cross-over is said to occur. The inference is that gas is present within the pores. In the case of the porosity logs from the Appin area, many of the sandstone units, most notably the Scarborough Sandstone, the Bulgo Sandstone and the Hawkesbury Sandstone are found to contain porosity cross-overs. A typical result for the Scarborough Sandstone is shown in Figure 3. Interpreted porosity is shown in blue and shale content in maroon.

Such analysis can only be used as a guide to the presence of gas. However, supportive results can be obtained from an analysis of resistivity logs and liquid hydrocarbons because these can be resistive zones within the geological section. As well, reports of gas make are often made while drilling the holes where gas cross-overs occur. In other Australian coal mining districts where strata gas is not present, the geophysical logs do not show the same behaviour in the porosity logs.

**GAS INDICATIONS FROM DOWNHOLE GEOPHYSICAL LOGS**

Geophysical logs from approximately 70 exploration boreholes in the Appin, West Cliff and Douglas areas have been studied to show strata characteristics and gas cross-overs. A comprehensive strata gas model is still under development but preliminary observations are as follows:

**Scarborough Sandstone**

This unit is 20-30 m thick and lies approximately 50 m above the Bulli Seam. The cumulative thickness of the gassy intervals may be up to 4 m, even when requiring a conservative difference of -0.5 % between the density porosity and the neutron porosity to be the threshold for a porosity crossover. Results contoured across this area are shown in Figure 4. It is notable that the thickness of the gas intervals decrease to the east where the underlying Wombarra Claystone is uniform and presumably is an effective seal. Over most of the area, there is approximately 1 m of cumulative gas, increasing to over 4 m in the north-western region where the geophysical log interpretation shows the Wombarra Claystone to be more sandy and more porous.

**Bulgo Sandstone**

This unit is approximately 150 m thick and can contain numerous intervals with porosity cross-overs. As shown in Figure 5, the cumulative thickness of the gassy sections is up to 20 m. (It reaches 30 m in boreholes S1780 and S1781 which are located on a sharp hill with 60 m local relief). There is also a tendency for the gassy sections to become more prevalent towards the top of the Bulgo Sandstone. This is consistent with the increase in porosity and sandiness within the Bulgo Sandstone that is observed from the geophysical log analysis and the notion that the overlying Bald Hill Claystone is an effective seal. However, understanding of the more detailed stratigraphic controls on the occurrence of the gas has still to be developed. The top of the Bulgo Sandstone is approximately 300 m from the Bulli seam and there is a reasonable expectation that the upper section of the formation will be isolated from the goaf.
Fig. 3 - Porosity cross-overs in the Scarborough Sandstone

Fig. 4 - Cumulative thickness of gas intervals in the Scarborough Sandstone
Resistivity logs

Interpretations of resistivity logs for a number of holes in the Appin area was undertaken by Mr Roland Turner of Borehole Logging Consultancy Services (Hatherly and Thomson, 2006). The interpretations were aimed at detecting hydrocarbons (gas and fluids) in the Scarborough and Bulgo Sandstones. Care was exercised to choose appropriate values of the resistivity for the pore water and the resistivity of clay rich formations and in general terms, the resistivity interpretations showed that the zones with porosity cross-overs were also resistive and with water saturations of less than one. This is taken to be independent confirmation of the presence of strata gas in the Scarborough and Bulgo Sandstones.

GAS INDICATIONS FROM SEISMIC DATA

BHP Billiton makes extensive use of 2D and 3D seismic reflection surveying to allow the detailed mapping of coal seams and the structures affecting them which might impact on mining. Another aspect of the seismic data that is receiving increased attention concerns the changes to the quality of the seismic signals which pass through the ground. The seismic velocities, the signal amplitude and its frequency content can all indicate the physical properties of the medium through which the waves are travelling. Wave velocity is related to the modulus and hence the well known empirical relationships between velocity and UCS. Amplitude and frequency content are related to the rate at which seismic signals disperse and are absorbed by the ground. In particular, if the rock pores contain gas as well as fluids, then higher rates of absorption occur.

The full waveform sonic and acoustic scanner logs can also show the same effects due to gas in the formation. However, care is required in identifying the causative factors affecting the seismic signal. Variations in signal strength in the borehole sonic logs can also be caused by changes in rock type. Gas issuing into the borehole and
mixing with the borehole fluids will also result in a significant loss of signal, even to the extent of no signal being detected at all. It is also a frequent observation that within about 200 m of the deep river gorges in this area, the seismic and sonic logging data are generally of poorer quality than elsewhere. Changes to the fold in the seismic data due to the inaccessibility for shot hole drill rigs and geological factors such as the lower water table and possible fracture systems, either pre-existing or induced by stress concentrations in the valley floors (Hebblewhite et al, 2000), are postulated to be the cause for these gorge effects.

An example of the effects of gas on seismic and full waveform sonic data is shown in Figure 6 (for locations see Figures 4 or 5). This seismic line is distant to the gorge systems and on the left (north) of the line there is a decline in the quality of the reflector from the Bulli Coal seam and the Scarborough Sandstone. Borehole S1728 is nearby and the results of the gas analysis for the Bulgo Sandstone in Figure 5 shows that the cumulative thickness of gas in the Bulgo Sandstone increases in this region. The decline in the quality of the seismic data is thought to be due to this.

Figure 6 also shows the relevant part of the full waveform sonic log and the geophysical log interpretation. In the Scarborough Sandstone at about 450 m depth, there is a reduction in the signal strength of the full waveform sonic log. This coincides with the inferred gas zone from the porosity log analysis. Further up the hole in the Bulgo Sandstone at 340 m - 350 m, a number of gas zones are indicated. Here the full waveform sonic log almost totally loses signal and the interpretation is that this is due to the flow of gas from the formation into the borehole.

![Fig. 6 - Full waveform sonic log and seismic section illustrating signal disruption to strata gas](image)
Figure 7 shows an example of the deterioration in seismic record quality that occurs when a seismic line approaches the gorge systems. For the left hand (western) half of this line, there is a marked reduction in the quality of the seismic data. The reflection from the Bulli Coal seam becomes irregular and the reflection from the Scarborough Sandstone is absent altogether. These effects are not interpreted to be due to changes in the nature of the reflectors. The interpretation is that these are due to the combined effects of the reduced seismic fold, the lower water table, stress and fracturing.

**Fig. 7 - Disrupted seismic reflectors in the Scarborough Sandstone**

**OIL OCCURRENCES**

Oil has been observed from drill core in the Narrabeen Group sandstone units overlying the Bulli seam. Minor oil occurrences have been observed in the 200 m interval above the seam within the Bulgo, Scarborough and Coal Cliff Sandstones. These formations consist of mainly of fine- to medium-grained lithic sandstones with minor pebbly conglomerates. Oil occurs predominantly in the coarser units. These formations consist of stacked alluvial channel sequences which are both laterally and vertically discontinuous. This provides an explanation as to why the oil is not restricted to specific mappable horizons. The reservoir appears to be confined to the Douglas Park Syncline.

Analysis of the oil indicates that it is terrestrial in origin and, most likely, originated from the adjacent coal seams and carbonaceous units.

Currently the oil is detected by visual observations of exploration borecore and drilling water sumps. An ultra-violet detector is also routinely used to detect the presence of hydrocarbons.

Samples within the strain relaxation envelope of longwall mining are subjected to chemical analysis to ensure that they do not contain hydrocarbons that are detrimental to the health of mine workers. Work to date indicates that the strata does not contain compounds which will impact on future operations. It is important to note that oil has only been observed in very small quantities and over short stratigraphic intervals.
DISCUSSION AND IMPLICATIONS FOR MINING

Faiz et al (2003) describe a model for the generation of coal seam and strata gas in the Illawarra region. It entails the following processes.

1. The generation of primary biogenic methane through the initial decomposition of plant material followed by the generation of thermogenic gases through normal coalification. These processes led to the development of mainly methane and minor amounts of CO$_2$ and ‘wet’ gases such as ethane. The thermogenic processes continued from the Permian period (250 million years ago) through to the Late Cretaceous period some 90 million years before the present.

2. Subsequent extension, uplift and erosion associated with the rifting of the Tasman Sea continuing through to the Early Tertiary period 50 million years ago. Large amounts of thermogenic gas were lost during this time however igneous activity, partly in association with the uplift and rifting events, simultaneously introduced CO$_2$ to the region. Because the uplift, erosion and extension also allowed the influx of meteoric water through open fracture systems, microbial agents were conveyed to the coal measures. These have subsequently acted on the CO$_2$ to produce secondary biogenic methane in significant quantities.

3. A present situation whereby the Illawarra region is under compression and only coals at depths greater than 700 m tending to contain thermogenic methane. The shallower coals contain mainly secondary biogenic methane and some CO$_2$ in areas where biogenic agents have not been introduced presumably on account of local variations in permeability. These areas also need to be shallower than about 600 m - 700 m because CO$_2$ is highly soluble at greater depths. In the areas where biogenic methane is present, higher gas saturation levels are typically encountered.

With such a geological history, it is evident that gases have had many opportunities to migrate into the strata overlying the coal seams. The existence of permeable pathways at some stage is an evident requirement but these pathways could be due to the intrinsic permeability of the sediments as well as via faults, dykes and fracture systems. The fracture systems might be those that allowed the escape of thermogenic gases from the coal and the introduction of the microbial agents, as well as those due to current stress effects around the gorges and due to mining itself. For the older pathways, it is quite possible that the development of the present dominant compressive stress field and diagenic processes have reduced the permeability of these zones.

In the absence of a clear model of gas migration into the overlying strata, the best strategy for understanding the occurrence of strata gas is to attempt to map it from the geophysical wireline logs. However the mapping becomes difficult if drillholes post-date mining induced changes to the gas contents (e.g. S1742). The correlation between the high levels of gas in the north west (particularly in boreholes S1780 and S1781) and an abrupt 60 m hill with associated capping of Wianamatta Shale is also intriguing but currently without explanation.

The converse, i.e. the flow of strata gas back into mine workings requires careful consideration. With a limited gas reservoir in the Scarborough Sandstone and a much larger reservoir in the Bulgo Sandstone, the important geotechnical questions is the proportion of these sources of gas that can be introduced to the mines via the fracture systems associated with longwall mining.

CONCLUSIONS

The use of downhole geophysical logs has provided a useful tool in identifying strata gas horizons.

Although downhole geophysical logs and seismic profiles can be used to determine the stratigraphic position and areal extent of the strata gas horizons, the following information is also required to determine the potential increase on mine gas emissions:

- Reservoir pressure
- The extent of mining-induced permeability subsequent to goaf formation
- The extent of fracture connectivity between the goaf and the gas reservoirs
There is a recognition that the discontinuous nature of the gas reservoirs and the localised and qualitative nature of the data-sets reduces the ability to accurately quantify the size of the reservoir.

Additional work is continuing to more accurately determine the extent of the overlying strata relaxation envelope in order to determine the extent of the connection between the gas-bearing horizons and the goaf. A related study to determine the fingerprint the gas composition in order to determine its stratigraphic origin is also on-going.

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The valuable contribution to this study of Roland Turner of Borehole Logging Consultancy Services is acknowledged.

REFERENCES

DEVELOPING METHODS FOR PLACING SAND-PROPPED HYDRAULIC FRACTURES FOR GAS DRAINAGE IN THE BULLI SEAM

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ABSTRACT: BHP Billiton Illawarra Coal is seeking ways to significantly increase gas capture rates from in seam drilling programs in its underground coal mining operations. Hydraulic Fracture Technology (HFT), a joint venture between SCT Operations Pty Ltd and CSIRO Petroleum, is working with Illawarra Coal to develop the capability to enhance gas drainage rates in the Bulli Seam using sand-propped hydraulic fracturing based on HFT’s experience at Dartbrook Mine where gas drainage rates were increased by 5 to 180 times.

One of the principal challenges for implementing sand-propped hydraulic fracturing in the Bulli Seam is the high vertical stresses that cause borehole breakout in horizontal holes drilled in coal. Borehole breakout effectively precludes the use of open hole straddle packers which are a convenient tool for placing multiple sand-propped hydraulic fractures in in-seam holes. Results of an initial six week trial undertaken at Douglas Project pit-bottom are described, which is aimed to developing the capability to install, grout and perforate casing so that straddle packers can be used for sand-propped hydraulic fracturing in overstressed boreholes. The primary goals of the pit-bottom trial at Douglas were to confirm that horizontal boreholes in Bulli coal at 500 m overburden depth are overstressed and unsuitable for use of open hole straddle packers, and to establish a method for installing, cementing and slotting casing so that straddle packers can be used to place hydraulic fractures. Both these goals were successfully achieved.

INTRODUCTION

Sand-propped hydraulic fracturing has been developed in the petroleum industry for stimulation of oil and gas wells, including coal seam methane wells. Several projects have been carried out in Australia and overseas with the objective of trialling stimulation of in-seam gas drainage holes by hydraulic fracturing in coal operations. A recent project conducted at Dartbrook Mine in 2002 (Jeffrey and Boucher, 2004; Jeffrey et al., 2005) demonstrated significant improvements in gas capture rates. In hard-to-drain coal the gas drainage rate was increased by up to 180 times, with the rate increasing over time.

The sand-propped hydraulic fracturing technique involves pressurising a section of borehole with water at sufficient pressure to overcome the in situ stresses in the ground and the tensile strength of the coal. Ideally a single vertical fracture develops and grows out from the borehole for a distance of 20-50 m in a direction parallel to the major horizontal principal stress. Sand propping involves the introduction into the fracture of specially graded sand suspended in the water flow. At the end of the treatment, the sand remains in the fracture and props the opposite sides of the fracture apart to provide a conductive drainage path back to the borehole. At Dartbrook, the in situ stresses were such that the horizontal boreholes remained stable enough for open hole straddle packers to be used to place the sand-propped hydraulic fractures. However, in the Southern Coalfield, the vertical stresses are typically too high for the borehole to remain stable and so the sides of horizontal boreholes drilled in coal are routinely overstressed. Failure of the coal caused by the overstressing results in borehole breakout, which enlarges the borehole in a horizontal direction. If borehole breakout is severe enough, it stops straddle packer systems from sealing and causes damage to the packers, thereby preventing their effective use, unless some sort of casing system can be developed that improves the surface against which the packers are set.

Borehole breakout is regarded as a major challenge to the effective implementation of sand-propped hydraulic fracturing in the Southern Coalfield. The work described in this paper is aimed specifically at finding suitable strategies to improve the condition of the borehole so that straddle packers can be used successfully. The concept of casing the hole is regarded as the approach most likely to be successful.

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A staged work program was developed with the intention of moving incrementally toward the ultimate goal of giving Illawarra Coal the in-house capability to place sand-propped hydraulic fractures. The stages of the work program involve:

1. Confirming the expectation that horizontal boreholes in coal are not suitable for setting straddle packers in the borehole without some sort of casing system.
2. Developing and testing suitable casing and cementing systems for deployment in overstressed boreholes.
3. Developing and testing a slotting system capable of perforating the casing to give access to the formation.
4. Testing a range of casing systems in an environment where the boreholes are subject to full overburden stress, and therefore likely to have broken out.

A trial site at Douglas Project pit-bottom was chosen as a convenient site for the casing, cementing and slotting tests. The site was located so that full overburden loading conditions were present and the trial did not cause undue disruption to the overall mining operation.

**CASING SYSTEM TRIALS AT DOUGLAS PROJECT PIT-BOTTOM**

The stages of the work program described in this section relate mainly to the Douglas Project pit-bottom trial, but some of the results from earlier casing trials conducted at CSIRO Petroleum in Melbourne are included where relevant.

**Site Description**

Figure 1 shows a plan of the Douglas Project pit-bottom trial site. The roadways in this area were developed some 25 years ago. A major geological fault structure is located immediately to the north-east of the site. The seam RL at the site is approximately 310 m below AH. The surface is approximately 190 m AH, so the overburden depth is nominally 500 m indicating a vertical stress of approximately 12.5 MPa. The seam thickness is nominally 2.2 m.

![Fig 1a: General location of Douglas site.](image)

![Fig 1b: Layout of boreholes at Douglas pit bottom.](image)
Six holes were drilled using a downhole motor at a nominal diameter of 96 mm. The intention was to drill these holes so that some were dipping down, some up and others with both up-dip and down-dip sections. The alignment of the holes was surveyed using the downhole survey tool. Figure 2 shows a summary of the hole alignments in plan and long section.

![Fig 2a: Plan of borehole alignments based on downhole surveying.](image)

Steel standpipes were installed in all the holes by the drilling crew. The standpipes were 9 m long in Holes 1 and 2 and 12 m long in Holes 3-6. The stand pipe used was nominally 4-inch diameter and had an internal diameter of 105 mm. A 4-inch BSP coupler was attached with a short BSP to Victaulic adapter fitted to the other end of the coupler. The cement head used during the trials was attached to the Victaulic fitting using a quick-set Victaulic clamp. Figure 3 shows a photograph of the site with the holes shown painted in red on the rib and numbered from 1 to 6.

**Caliper Logging**

Holes 2-6 were logged using the eight arm caliper logging system developed in ACARP Project C12021 (Jeffrey et al. 2005). Two of these holes were blocked with coal debris. When Hole 1 was flushed out ahead of caliper logging, the hole was blocked a short distance beyond the end of the casing and large chips of coal were observed coming out of the hole. This hole was not logged because the risk of the caliper tool becoming jammed in the hole was considered to be too high.
Figure 4 shows the results of caliper logging in each of the holes. The caliper was logged continuously as it was retracted along the hole, but the depth was only measured every 0.1 m. In most of the holes the instrument was able to be oriented within 15° using a downhole tilt indicator, so that the arms were oriented vertical, horizontal, 45°-225° and 135°-315°. The maximum and minimum values measured on any of the arms over each 0.1 m interval are shown as well as the average value measured on each arm.

The caliper logs show that the boreholes have a horizontal diameter greater than the as-drilled diameter over a substantial length of the hole. The horizontal diameter is generally greater than the vertical diameter and ranges up to about 180 mm in some sections. The vertical diameter is typically less than the nominal diameter of the hole. This may be a result of closure due to borehole breakout, but it may also be a function of debris in the bottom of the hole.

The key result of the caliper logs is that the boreholes are unsuitable for using open hole straddle packers for hydraulic fracturing without some sort of casing system. If straddle packers were used in these boreholes without casing, the packers would not be expected to provide an effective seal at the treatment pressures of 10-15 MPa and the packers would rupture on a regular basis.

**Casing Selection**

Three types of casing were selected for testing for the pit-bottom tests at Douglas Project; 76 mm outside diameter steel casing with a 3.5 mm wall thickness, 85 mm outside diameter fibreglass casing with 8 mm wall thickness and 84 mm outside diameter PVC casing with 6 mm wall thickness. Four holes were cased using steel casing, one with fibreglass and one with PVC casing. The steel casing was supplied in 3 m lengths and externally coupled. The PVC casing was supplied in 6 m lengths with upset ends. The 3 m long fibreglass casing sections had integral threads and upset ends.

The main criteria for casing selection relate to ease and robustness of handling, ability to resist external collapse pressures during grouting and hydraulic fracturing, cuttability by the longwall shearer and ease of separation from the coal product stream once it has been cut. Thin walled steel casing has advantages in all these areas except perhaps cuttability and is the preferred option for routine use.
Fig 4: Caliper measurements from Holes 2-6 at Douglas.
Cement System

The cement system used in the trial was designed by Schlumberger Oilfield Services in Perth. They carried out tests to establish the optimum mixing ratios for the locally available general purpose Portland cement used in the trial. An antifoaming agent, a dispersant and a gas block agent were used in the mix. Mixing is carried out by adding the liquid additives to the mix water and stirring well. The cement is then added to the liquid as stirring continues so that no lumps form. The cement is pumped into the casing and returns up the annulus outside of the casing. When sufficient cement has been pumped to fill the annulus, a displacement plug is placed in the casing at the collar of the hole and pumped down the hole with water. The displacement plug acts as a barrier separating the cement from the water. When it reaches the end of the hole, there is no grout inside the casing, and provided there are no substantial loss zones, the grout should have returned up the outside of the casing to the collar of the hole.

Perforation System

Once the casing system has been installed, it is necessary to gain access to the coal seam from inside the casing string by perforating the casing. There are various systems available ranging from explosive perforators to mechanical cutting devices. The system that is most compatible with the equipment used for sand-propped hydraulic fracturing is an abrasive slurry system based on a sand-water system injected through a jet at high pressure onto the casing. This slurry jet cuts a slot in the casing, through the cement and into the coal within a few minutes (depending on the pressure available). By rotating the tool, a circular slot can be formed. By withdrawing the tool, a linear slot is formed.

A slotting tool was developed, tested and modified at CSIRO Petroleum in Melbourne. The tool jets were sized so that the steel casing could be cut in about 4 minutes. PVC and fibreglass casing required less time to cut than steel. Figure 5 shows a section of steel casing cut under these conditions. For this test, a short section of casing was encased in a concrete block and then cut using a sand-water slurry pumped at 120 lpm and 13.79 MPa (2000 psi) for 4 minutes. The openings shown in Figure 5 would provide more than enough access to the coal seam for carrying out hydraulic fracture stimulations. Higher cutting pressures are expected to reduce the slotting time further.

Fig 5: Slot cut into steel casing in yard test, using abrasive slurry.

Casing Installation

The Douglas pit bottom work program provided an opportunity to trial methods to place and cement casing into medium length holes, perforate the casing using an abrasive slotting tool and test the integrity of the hydraulic seal provided by the cement. The hydraulic conductivity of the coal was characterised by carrying out injection/falloff well tests. The procedure followed for each hole was generally as follows:

1. Run an HQ-size bit on the end of a drill string into the hole while rotating the bit and flushing with water. This cleaning operation ensured the hole was at least 96 mm in diameter and removed debris that may have collected in the hole since drilling.
2. Run the casing string into the hole. A casing shoe valve was installed on the first length of casing run to act as a check valve – allowing fluid to pass from the casing into the hole but not in the opposite direction. Typically the casing could be pushed in by hand for 25 m or so and then was pushed by the drill rig for the rest of the hole length (50 m).
3. Install the cement head on the stand pipe and connect the casing into the cement head so that the annulus and the inside of the casing were isolated from one another. The hole was then filled with water from the pit water supply, allowing the system to be checked for hydraulic integrity.
4. Mix and pump a volume of cement into the casing. After the first hole tested, the volume of cement mixed was standardised at 450 litres per hole. This was pumped into the casing and then the displacement plug was pumped down the casing to push the cement out into the annulus.
5. Wait 4 to 12 hours for the cement to cure. Then remove the cement head.
6. Test the cement hydraulic seal by notching the casing and then injecting fluid through the notches while monitoring for leakage along the cement filled annulus.
7. Carry out coal characterisation tests as required.

CHARACTERISATION TESTING

Two types of characterisation tests were used at Douglas.

Step-Rate Tests
The step-rate test involves injecting into the entire hole or into a single slot at a constant rate for a fixed period of time. The rate is then increased to a higher fixed rate and injection at that rate continues for the same fixed period. Typically 4 to 8 steps in rate are used for one test.

When injecting into the entire hole, the test results give an estimate of the injection pressure versus injection rate response for the hole. This test is useful in estimating what the rate of loss of a particular fluid (for example, cement or water) will be into the coal if the hole is pressurised to some fixed level. A pressurised hole will lose fluid at a decreasing rate if the pressure is held constant, but this test provides a first order estimate of the loss rate for short pressurisation intervals (such as during cementing).

As a fracturing response test, the step-rate test is used to estimate the fracture extension pressure. The fracture extension pressure is the pressure required to extend a hydraulic fracture and is an upper estimate of the minimum principal stress magnitude.

Injection/Falloff Well Test
In the injection/falloff well test, water is injected through a slot at a constant low rate such that the pressure rise does not approach the fracturing pressure. A short test involves 1 to 2 hours of injection followed by 1 to 2 hours of falloff. A more standard test in coal would require 4 to 6 hours of injection and up to 12 hours of falloff. Short tests were used at Douglas to see how these tests might be carried out in a production setting and to estimate the coal parameters.

The data from an injection/falloff well test are analysed to obtain the permeability of the coal and the initial pressure in the coal. In addition, a parameter called the skin is obtained which is an indication of the permeability damage or stimulation effect existing locally at the borehole.

RESULTS

All six holes at the Douglas pit bottom site were cased during the project although it was not possible to fully install casing in Hole 1 because of hole instability. Casing strings were installed and cemented in the other five holes.

In general, there were no difficulties installing the casing, provided the holes were flushed immediately beforehand using a 96 mm drill bit and drill rods. The casing was able to be pushed into the hole by hand to about 25 m before it was necessary to use the machine. However, without cleaning the hole first, the casing was typically not able to be run much past the end of the standpipe.

Cementing through the casing with return along the annulus proved to be more challenging, mainly because of equipment limitations. The cement system was able to be mixed without difficulty in two high-shear mixing tubs.
The cement was then pumped into the hole using a 2 MPa progressive cavity pump. In several of the holes, grout return was observed at the collar of the hole. But power outages, blockages and insufficient pump pressure meant that in several holes, the displacement plug was not able to be pumped to the end of the casing. Once the steady flow of grout was interrupted, it proved difficult to get it restarted again with the equipment available.

A small gap was observed to occur at the top of the cement annulus in some of the holes. While the existence of a small gap, of width less than about 1mm, is not necessarily a problem for subsequent treatments using sand propped hydraulic fracturing, it is nevertheless an inconvenience that would be best to eliminate.

If a gap is present, the casing string needs to be strong enough to withstand the external fluid pressure when injection fluid leaks through the gap along the outside of the casing. The loading on PVC, fibreglass, and steel casing required to cause collapse is about 0.3 MPa, 3.5 MPa and 13 MPa respectively. Steel casing is therefore the preferred option from the perspective of collapse pressure at treatment pressures expected to be of the order of 10 MPa. Fracturing through slots in the casing can occur even if there is some fluid loss into the gap. Once sand is started, it will bridge across and plug any small gaps.

The gap at the top of the casing is caused by various processes.

- The cement may settle slightly during curing and free water rises to the top. If this occurs, the free water leaves a small gap along the top of the hole. Careful mixing of the cement and use of dispersants limits the amount of free water in the mix.
- If the displacement fluid (typically water) bypasses the displacement plug or leaks from any of the casing couplers, it can channel back along the casing, preferentially at the top as it is of lower density, and form a gap or further enlarge any gap that may already exist.
- Fluid pressure in the seam can drive water (or gas) into the hole after cementing. This fluid will tend to separate at the top of the hole. Maintaining pressure in the cement that is above the pressure in the coal will eliminate this problem.

By pumping grout into the gap at the collar, it was found to be possible to effectively seal up any gaps that had formed, but there is clearly benefit in not having any gap form in the first place. Pumping cement into a slot placed some distance along the casing could also be used, but was not attempted during the Douglas trials.

Slotting of the casing using the abrasive jet proved to be relative easy. With the pressures available, it took several bags of sand and some 4-10 minutes of pumping to cut a circular slot in the steel casing. Higher pressures and better control of the rotation rate is expected to reduce this timeframe to just a few minutes in normal operation.

**STEP-RATE TEST RESULTS**

A step-rate test along the full length of the hole was carried out in Hole 6 using water. The hole was first filled with water from the pit-water supply. The step-rate test was then carried out by injecting into the casing (before it was cemented) through the cement head. The return valve on the annulus was shut during the injection so the water injected was forced into the seam. The highest rate achieved was 55 litres per minute and the pressure during this step reached 2 MPa.

The water test establishes a reference point for comparison to the step-rate test carried out in Hole 2 using cement. After starting the cementing and obtaining cement returns through the annulus at the collar to confirm that the entire hole was full of cement, the annulus return valve was closed and injection was carried out in ever-increasing rate steps. The final step was at 6 litres per minute and the pressure reached 1.8 MPa. The cement loss rate at 1.8 MPa was therefore 6 lpm. It is anticipated that this rate would decrease with time.

The fluid pressure in the seam at this site is very low because the coal contains virtually no gas and has been draining into the mine openings for about 25 years. Therefore, the 1.8 MPa is acting against essentially no hydraulic pressure in the seam and represents a pressure above the seam pressure.

In a gassy area, the seam pressure might be 3 MPa, but to force cement into the seam the pressure would have to be increased above 3 MPa and if the cement pressure were increased to 4.8 MPa the loss rate might be about 6 litres per minute per 40 m of hole (allowing for the stand pipe). The intention in gassy areas is to keep the cement...
pressure just above the gas pressure in the seam so the loss of cement under these conditions would be expected to be less than 6 l pm per 40 m of hole.

A short well test was conducted through one of the slots at 25 m in Hole 4 after re-cementing of the annulus. The well tests provide an order of magnitude estimate of seam permeability and pressure at the site. Water was then injected for 1 hour at 1.47 litres per minute. The falloff in pressure after shutting in the slot (isolating the pressurised fluid in the slot and allowing it to drain away into the formation) was then recorded for 1 hour.

Figure 6 shows the analysis of the falloff data from the well test in Hole 4 at the 25 m slot. This analysis is considered to give the best estimate of reservoir pressure. Figure 7 shows a second analysis plot of the falloff data for the well test in Hole 4 at the 25 m slot which gives the best estimate of permeability.

The flattening of the falloff curve in the Horner plot is indicative of a test carried out by injecting water into an unsaturated seam. The early part of the curve (larger Horner time) where the slope is greater is used to obtain an estimate of the seam permeability. In later times (small Horner time) the pressure is affected by the low gas pressure in the reservoir and the slope tends to flatten and approach the reservoir pressure asymptotically. The intercept of this portion of the curve with a Horner time of 1 provides an estimate of the reservoir pressure. If more accurate estimates of permeability and reservoir pressure are required, the injection and falloff portions of the test should be extended to several hours each.

Notwithstanding the short duration of the tests, permeability of about 5md and a reservoir pressure of about 34 kPa (5 psi) seem reasonable estimates based on the site location. It should also be noted that this analysis assumes radial flow conditions exist while such a short test is expected to take place almost wholly in a spherical flow period.
CONCLUSIONS

The primary goals of the pit-bottom trial at Douglas were to:

- Confirm that horizontal boreholes in Bulli coal at 500 m overburden depth are overstressed and unsuitable for straddle packers without a casing system.
- Establish a method for installing, cementing and slotting casing so that straddle packers can be used.

Both these goals were achieved.

Caliper logging showed that horizontal gas drainage holes drilled in coal at 500m overburden depth are subject to vertical stress sufficient to cause large amounts of borehole breakout and compromise the effective use of straddle packers in open holes. The horizontal diameter was often larger than the 96 mm nominal hole size and in some sections of the holes reached 180mm indicating considerable breakout from vertical overstressing of the borehole.

Three different kinds of casing, steel, fibreglass and PVC were successfully installed, cemented and then slotted using an abrasive jet system. Of these, steel was confirmed as the most suitable for routine use because of its superior strength and collapse properties, and the ease with which it can be separated from the coal product stream using magnets.

A petroleum well cement system was used. The cement contained a gas block additive which helps resist gas from entering and channelling through the cement before it cures.

The cementing operations at Douglas Project pit-bottom have proved the concept of using a cement head and a displacement plug system to place the cement into the annulus.

Steel and fibreglass casing were notched successfully using an abrasive jet slotting method. Increased pressure during slotting is expected to reduce the time to cut slots in steel casing to just a few minutes. The sand used for slotting was carried out of the hole to the collar in the returned water and the slotting tool was easily moved further out of the hole after each slot was cut.

The PVC string was cemented in with pressure inside the casing exceeding 4MPa without bursting the casing. The casing did not collapse over the 17 m length that could be inspected from the collar. Likewise, the fibreglass casing withstood the cementing pressures without damage and packers were run to 35 m with no difficulty. However, setting packers to pressures above 20 MPa, anticipated for placing sand-propped fracture, may not be possible in these lower-strength casing materials. Steel is by far the strongest casing material used and can withstand high external and internal pressures without damage.

There does not appear to be any reason why this method of casing installation, cementing and slotting would not be effective as a practical means for allowing sand-propped hydraulic fracturing to be placed in overstressed coal holes using straddle packers.

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REFERENCES


**SURFACE GOAF HOLE DRAINAGE TRIALS AT ILLAWARRA COAL**

Tim Meyer

**ABSTRACT:** Surface goaf gas extraction methods are successfully applied at a number of Australian and overseas underground coal mines. BHPB Illawarra Coal has recently undertaken a trial program to determine the effectiveness of surface goaf wells to reduce gas concentrations within the longwall ventilation circuit, and minimise gas related production delays. Three trial wells have been completed within the Bulli Seam operations. Considerable production variation between the three wells was recorded. A goaf gas reservoir model is discussed which describes a sequence of permeability changes within the different goaf strata, dictated by stress changes associated with caving, then recompaction. A requirement for further work is identified to improve understanding on the 3-D properties of the goaf in terms of permeability variations and pressure distributions. An overview of the surface goaf extraction trials including descriptions of the gas plants and well production results is provided.

**BACKGROUND**

Goaf gas typically originates within the seams and strata surrounding the extracted working seam. Gas is liberated from these seams/strata by the processes of de-stressing and comminution associated with longwall caving. The liberated gases migrate along newly created fractures into the goaf voids. Goaf gas drainage holes attempt to remove these gases from the goaf before they can flood into the face and return circuits. Goaf drainage is therefore considered a post-drainage technique.

Surface goaf gas extraction methods are successfully applied at a number of Australian and overseas underground coal mines. Review of the techniques used indicates a wide variation in the design of the gas extraction plants, and the spacing and location of the goaf gas wells. Factors that influence these parameters include the specific gas emission, gas composition, longwall panel width and mining rate, depth of cover and goafing characteristics.

BHPB Illawarra Coal has recently undertaken a series of trials of goaf gas extraction wells drilled from surface. The aim has been to determine the effectiveness of these wells in reducing the gas quantity reporting to the longwall face and returns, and in doing so to reduce the number and severity of gas related production delays. To date, there have been three trial wells – two at West Cliff Colliery and one at Appin Colliery.

The general scope of the trial project was as follows:

- Drill and complete three wells, cased with cemented 250 mm ID thick-walled steel pipe to below the Bulgo Sandstone formation, with uncemented 200 mm ID slotted casing installed to a short distance above the working seam
- The procurement (lease), installation and commissioning of two goaf gas extraction plants
- Continuous operation of the plants for an extended period of time, defined by the period for which the plants are noticeably beneficial to underground conditions
- Plugging and abandonment of the wells upon completion of gas extraction
- Review of the data to determine the effectiveness of the technology

An overview of the surface goaf extraction trials including descriptions of the gas plants and well production results is presented. A description of the likely mechanism for gas release and migration into the goaf space is provided.

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1 BHP Billiton Illawarra Coal
GAS RESERVOIR CHARACTERISTICS

Some of the main considerations in the design and implementation of a surface goaf well program are:

- the reservoir properties of the gas bearing formations and seams lying above or below the extracted seam section,
- the effect that extraction process has on these reservoirs; primarily pressure and permeability changes,
- the characteristics of the goaf and surrounding strata in terms of their resistance to the flow of this gas from entering the goaf volume.

Gas reservoir properties are provided in this section. Hypothetical descriptions on the likely effect that coal extraction has on these reservoirs, and the flow characteristics of the goaf zones, are provided later on in the generalised goaf gas reservoir model.

A typical stratigraphic profile for the West Cliff goaf well trial area is given in Table 1, along with estimates of the potential contribution from each of the major gas bearing strata to the specific gas emission associated with production (Moreby, 2005). Table 1 illustrates that potentially up to 45 m$^3$ of gas is liberated for every tonne of coal extracted. Management of this high specific gas emission has to date been by a combination of techniques including bleeder airways drawing this gas towards the tailgate and inbye end of the goaf, capture of floor seam gas emissions by cross-measure drilled boreholes, as well as goaf drainage holes drilled from adjacent roadways. This trial is the first significant attempt by BHPB-IC to drain goaf gases from surface drilled holes.

No accurate measurements of the roof stratum gas contents exist. This is primarily due to the fact that this gas exists as free gas (or in solution with water) within the pores of the sandstone matrix. Any attempt to recover core from this rock inevitably results in the core becoming de-pressurised, and hence losing a large part of this gas before it can be captured for measurement.

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In determining the potential contribution from the roof stratum presented in Table 1, assumptions were made that the gas is only contained in the lower sandstone bearing formations (Bulgo, Scarborough and Coalcliff), that the gas occurred as free gas at pressure, and that the unsaturated porosity of the stratum was 1%. The potential
emission from each stratum was then adjusted by the Flugge method to account for the likely release based on panel width and vertical separation to the working seam.

Potential floor seam gas emissions listed in Table 1 were calculated based on desorption from 100% CH4 fully saturated gas content values (around 14 m³/t) to residual values based on hydrostatic pressure conditions existing below the extracted section. These amounts were then adjusted by the Flugge method to account for panel width and depth below the extracted seam.

Actual estimates of cumulative specific gas emissions for Appin Longwall panels 402 to 405 are presented in Figure 1, (Self 2004). SGE is calculated based on totalising gas reporting to the ventilation circuit (tailgate and bleed), gas captured in cross-measure drilled holes into the floor seams (which tend to only flow after the longwall has passed) and goaf drainage holes, and dividing by the tonnes produced from the longwall panel. The graph illustrates that specific gas emissions of between 35-40 m³/t were measured for these four panels. This compares reasonably closely to the predicted quantities indicated in Table 1.

\[
\begin{align*}
\text{Day} & \quad 0 & 5 & 10 & 15 & 20 & 25 & 30 & 35 \\
\text{m³/t} & \quad 5 & 10 & 15 & 20 & 25 & 30 & 35 & 50 \\
\text{LW402} & \quad \text{LW403} & \quad \text{LW404} & \quad \text{LW405}
\end{align*}
\]

**Fig. 1 - Cumulative Specific Gas Emission - Appin Longwall Panels 402–405**

**WELL DESIGNS**

A generalised well design typical of all three trial wells is shown in Figure 2. The main design features are as follows:

- 14” surface casing installed and grouted to around 50 metres below GL.
- 10”, thick walled, welded line pipe grouted to at least five metres below the floor of the Bulgo sandstone formation.
- 8” slotted casing installed without rigid connection (floating) to between 5 and 35 metres above Bulli Seam roof (50 mm circular slots at a density of ten per six metre length).

Actual details of installed wells are in Table 2.

Figures 3 and 4 show the location of the trial wells at West Cliff Colliery and Appin Colliery, respectively.
Fig. 2 - Typical trial surface goaf well

Table 2 - Details of installed goaf drainage wells

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<td>Base of Stanwell Park Claystone</td>
<td>425</td>
<td>453</td>
<td>435</td>
</tr>
<tr>
<td>Base of Scarborough Sstone</td>
<td>451</td>
<td>479</td>
<td>458</td>
</tr>
<tr>
<td>Base of Wombarra Shale</td>
<td>486</td>
<td>514</td>
<td>493</td>
</tr>
<tr>
<td>Base of slotted (sliding) 8” casing</td>
<td>470</td>
<td>523.5</td>
<td>504</td>
</tr>
<tr>
<td>Bulli seam roof</td>
<td>505</td>
<td>533.5</td>
<td>514</td>
</tr>
</tbody>
</table>
Fig. 3 - Aerial view showing location of WCC SGW’s Nos 1 & 2

Fig. 4 - Aerial view showing location of Appin SGW No 1
GOAF GAS EXTRACTION PLANT No 1 DESCRIPTION

Overview

Goaf gas extraction plant No 1 is a modified unit leased from Anglo Coal Australia, originally designed for operation at the Dartbrook Mine, Muswellbrook NSW.

The plant consists of a Howden centrifugal fan, 1.38 m in diameter, driven by a 150 kW 4-pole electric motor. The motor and fan are housed in an acoustic enclosure. An 110 kW variable voltage – variable frequency (VVVF) drive is used to power the fan motor. The under-rating of the drive (compared to the motor) is possible due to the lower flow rate expected from the WCC well. A 150 kVA 415 V genset supplies power to the goaf extraction plant and associated control systems. Extracted gas is exhausted to atmosphere through a vent stack approximately 8 metres above ground level.

The plant, as originally designed, has the capacity to extract in excess of 2000 litres per second with a nominal suction pressure of 10 kPa. Dartbrook has previously achieved these flows by connecting up to three individual wells to a single gas plant. Due to the fact that only single wells will be connected to the plant during the BHPB-IC trials, and considering flow restrictions caused by the diameter and length of the casing installed into the WCC goaf well, the plant is expected to extract between 500 and 1000 lps per well.

A comprehensive range of electronic sensors are used to monitor important operating parameters of the plant. The outputs from these sensors are connected to a TROLEX Sensor Controller module, which allows user-defined set-points to be programmed. The TROLEX Sensor Controller has four relay trip outputs, three of which are connected to latch relays which cut-off the VVVF drive output, the fourth is connected to the genset fuel solenoid, therefore cutting power to the entire gas plant. The fan speed is manually adjusted by a potentiometer connected to the VVVF drive.

The plant is connected to the well via a ten metre long 14” victaulic pipe range. At the wellhead is a 250 mm ANSI 300 lb gate valve, used as the main isolation valve. Above this are a 90 degree elbow and 10” butterfly valve, then an adapter to the 14 in victaulic pipe line. The gas flow passes through a flow measurement venturi device and the flame arrestor before entering the intake of the fan. The full plant and site setup for West Cliff Surface Goaf Well No1 is shown in Figure 5.

Fig. 5 - Goaf gas extraction plant No 1
Plume Ignition Protection

Lightning presents the greatest risk of ignition of the vented gas plume. A design for lightning protection, in compliance with Standard NZS/AS 1768 was commissioned. This work highlighted that in order to protect the gas plant infrastructure from a direct strike, 4 x 15 metre high lightning masts would be required, spaced around the gas plant and wellhead, with each mast individually grounded by an array of four earthing electrodes grouted in to a depth of five metres. In order to prevent lightning from entering the probable gas plume envelope, the lightning masts would need to be increased to over 25 metres in height. On reflection that the design standards do not prevent the possibility of a direct strike, but merely reduce it by a factor of 95%, it was decided that a better form of lightning ignition prevention control was to implement a procedure which required the plant to be shut down whilst storm activity was within a 10 km radius of the gas plant.

In addition to the above procedure, the gas plant has been fitted with other devices to minimise the hazards associated with gas plume ignition. A heat detection “pyro-tube” was fitted above the vent stack. This tube reacts to heat from plume ignition, causing the pressure in the tube to substantially rise. This pressure rise is detected by a mechanical regulating valve fitted onto a large bottle of compressed CO$_2$. The regulating valve fully opens, dumping CO$_2$ into the vent stack. At the same stage, a pressure sensor detects activation and sends an electrical signal to operate a flag relay, which in turn trips the VVVF drive. Flame arrestors were fitted to the vent stack and to the inlet side of the gas plant, situated between the fan and the wellhead.

GOAF GAS EXTRACTION PLANT No 2 DESCRIPTION

Overview

Goaf Gas Extraction Plant No 2 is based on a Nash CL3002 liquid ring vacuum. The pump is powered by a constant speed 110 kW four pole electric motor, connected by a V-belt drive system that reduces the pump speed to about 350 rev/min. At this speed the plant has capacity to draw in excess of 1200 litres per second at suction pressures up to 60 kPa. The motor, V belts, pump, and control panel are mounted on a common skid. A 375 kVA genset is required to power the DOL start 110 kW motor, as well as other site electrical requirements.

Control of the plant is provided by a small PLC unit. The PLC monitors the state of a compressed air automatic valve shut-off system, gas composition monitoring equipment and the liquid ring pump seal water supply system. Signals from all three systems must be healthy for the PLC to allow operation of the vacuum pump.

The compressed air automatic valve shut-off system is based on two pneumatically operated butterfly valves, one positioned at the wellhead, the other on the discharge line. These valves, configured to open when energised, will automatically close if the gas plant is tripped or manually stopped. A small air compressor supplies compressed air through a solenoid activated 3/2 way valve, configured to dump the air pressure if de-energised. The PLC monitors the air reservoir pressure, to ensure sufficient pressure is available to open the valves.

Gas composition monitoring is achieved by Trolex CH$_4$ and O$_2$ sensors, feeding into a Trolex Sensor Controller unit. The relay output from this unit is connected into the PLC input to register gas out-of-range conditions and trigger a shut-down of the vacuum pump.

Two detonation arrestors are installed in parallel after the main isolation valve as protection against flashbacks if other protection systems fail. Each of the arrestors can be individually isolated by manually closing its inlet and outlet valves and can then be removed for cleaning the elements. A non-return valve is installed after the arrestors to provide additional protection against backflow down the borehole. A 10 in diameter steel Victaulic pipe is run between the wellhead and the vacuum plant. A venturi flow measurement device is fitted near the plant to provide flow rate measurements. The plant, inlet pipework and flow separator can be seen in Figure 6.

The overall gas flow rate is varied by recirculating a portion of the gas from the discharge of the separator back to the inlet of the vacuum pump, by manual operation of a butterfly valve on a recirculation pipe. Because the gas is cooled by the flow of seal water the entire throughput of the pump can be recirculated if necessary without overheating.
ENCLOSED GAS FLARE UNITS

An important aspect of the Goaf well Trial was to investigate the applicability of enclosed flares as a means of disposal of extracted gas. Not only is there considerable environmental benefits in flaring the gas as compared to free venting to atmosphere, there was serious concern that the odour of the exhausted gas would be detectable by local residents in close proximity to the selected trial sites. This close proximity also precluded open flaring options, from a visual impact perspective, hence the need to use enclosed flares.

Two enclosed flare units were hired from Landfill Management Services Pty Ltd (LMS). Refer Figure 7. As their name suggests, LMS specialise in enclosed flare units for the flaring of landfill gas. The flare units are essentially a refractory lined stainless steel stack approximately eight metres high and 1.4 metres in diameter. A small centrifugal fan in each unit is capable of drawing up to 1000 m$^3$/hr of gas at around 15 kPa suction pressures. As gas was being supplied at pressure by the goaf plants to the flare units, these flare fan units were disabled.

The supplied gas is injected into the base of the stack through a series of burners. The combusting gas/air mixture rises up the enclosed stack, drawing air in through a series of vanes at the base of the stack.

Numerous monitoring and safety devices are fitted to each flare unit, including:

- Draegar Polytron CH4 sensor
- Stack flame detector (UV light)
- Flashback temperature sensor
Output from these devices is monitored by a small PLC unit, which will trip a solenoid activated shutoff valve if threshold levels are reached. Additional protection from flashback is provided by a flame arrestor in the discharge pipeline.

**OPERATING PROCEDURES AND MONITORING**

In light of the short-term duration of the trial program (approximately 6 months), a decision was made not to invest in automation and telemetry systems that would enable remote control and monitoring of the plants. Instead, the gas plants were supervised on a continuous 24/7 basis. Similar monitoring regimes had been used for monitoring of Bulgo drainage holes on the Appin mine lease.

A set of “Normal Operating Procedures” was developed which prescribed the sequence of actions required for plant start-up and shut-down. In addition, a series of specific operating procedures was developed for instances when operating parameters reach respective trigger levels, as defined in the TARP. These are termed “abnormal operating procedures” because they were only applied when particular sensor readings fell out of normal range. Depending on which sensor measurement reached a trigger level, a corresponding procedure sheet was to be referenced to specify appropriate procedures to be followed by the site monitor.

The monitors’ duties included taking regular readings of all the sensors and monitors around the gas plant, and recording these onto a paper shift monitoring report, as well as entering this data into an Excel spreadsheet for daily electronic distribution to relevant personnel.

**WEST CLIFF COLLIERY SGW NO1 RESULTS**

**Initial Connection**

The first trial well, WCC SGW No1 was situated above West Cliff Longwall 31, approximately 715 metres outbye from the face installation road. The well was situated 40 metres from the tailgate drive. Refer Figure 3.
The longwall progressed under SGW No 1 on 10/12/05. No sign of connection between the goaf and the well was seen until the 11/12/05, when the longwall had advanced approximately ten metres past the well. At this point, the pressure at the well head dropped over a period of a few hours to around -75 kPa. This high suction pressure was due to the column of water which was originally in the well slowly emptying into newly connected goaf voids. This high vacuum pressure was sustained for around 24 hours, indicating only a very slight leakage path, after which it gradually reduced back to around 0 kPa over a 12 hour period.

The wellhead pressure was constant at around 0 kPa for the next 48 hours, then on the morning of the 16/12/05, a slight positive pressure was measured. At this stage the longwall had progressed to approximately 50 metres past the well. A plot of wellhead pressure versus time for this initial connection period is shown in Figure 8.

![Fig. 8 - Plot of wellhead pressure and longwall position relative to well](image)

**WCC SGW No 1 – Production History**

The goaf gas extraction plant was run continuously from 16/12/05 to 6/3/06. A full plot of plant suction pressure, measured gas flow rate and face position relative to the well is shown in Figure 9.

Initial gas flow rates of around 600 lps were achieved with a suction pressure of 7 kPa. After an initial “running in” period of a few days the motor frequency was turned to 70 Hz, producing the maximum allowable fan speed of 2100 rev/min. Suction pressure increased to 9 kPa and a flow increase to 700 lps was achieved. At this stage the longwall was approximately 110 metres past the well.

On the 27/12/05, with the wall around 150 metres past the well, gas flow rate began to climb steadily over a three day period, peaking at a maximum of 1000 lps, but then began to decline. Coinciding with this peak was a water release event, leading to a fine mist emanating from the stack. It is probable that these events were caused by casing breach in the Bulgo Formation, with an associated inflow of pressurised gas and water. Following this spike in gas flow and water event, the gas flow gradually dropped over a ten day period to around 500 lps. Following this, gas flow averaged 480 lps for the remainder of the plant operating time, up to 6/3/06.

The main indicator of effectiveness for a goaf well is the reduction in gas reporting to the ventilation circuit. Figure 10 shows the gas level in the tailgate immediately prior and after commencement of the goaf gas extraction plant. Within 40 minutes the tailgate methane concentration had reduced by 0.8 %. A more detailed analysis of the tailgate and bleeder circuit gas levels over the ten week period for which the plant was operational, indicates that the effect on tailgate gas percentage diminishes with distance. For instance, Figure 11 presents tailgate gas levels during a shut-in of the well for a brief three hour period when the longwall was 270 metres past the well. An obvious increase in tailgate gas level of approximately 0.25 % is evident coinciding with the shut-in. This rise is negated within 90 minutes of re-starting the gas plant.
Free Venting

During scheduled maintenance shutdowns of the plant, it was observed that the well would continue to free vent gas at considerable rates (in excess of 400 lps), and that shutting the main wellhead valve caused the wellhead pressure to rise in excess of 70 kPa. Figure 12 shows the free venting flow rate and shut-in pressure for the well during a typical shut-down.

Whilst providing an insight into the characteristics of the gas reservoir above the goaf, it also highlights that the well connection to the goaf was now substantially restricted as demonstrated by the high pressure build-up in the well casing. Notwithstanding this poor connection, shutting in the well impacted on the longwall gas makes as noted by significant increases in tailgate gas levels coinciding with the shut in periods (between 0.7 % and 0.2 %).

Based on these high free venting flows and the continued positive impact on tailgate gas levels, a decision was made to continue free venting from WCC SGW No 1 after the plant had been relocated. On the 6/3/06, the plant was shut down and mobilised to WCC SGW No 2 site. A free venting facility was established with the inclusion of a pneumatically operated shut-off valve and detonation flame arrestor. An eight metre high 10 in diameter vent stack was situated approximately ten metres from the wellhead. A compressed air line was positioned above the vent stack in case of ignition – the tube would burn, releasing the compressed air and the pneumatic shut-off valve would close. Trolex CH₄ and O₂ sensors were fitted to monitor gas composition. Free venting was maintained for the period 10/3/06 till 1/5/06, during which the average gas flow from the well was in excess of 320 lps, at a gas purity of around 90 % methane.
Fig. 10 - WCC SGW No 1 reduction in tailgate gas on plant start-up

Fig. 11 - WCC SGW No1 Effect of plant off/on at 270 metres in bye of longwall
Regular bag samples were collected from the WCC SGW No1 flow since extraction commenced on 16/12/05. Figure 13 shows gas composition history from well start-up to mid April. Evident in the graph is the fact that methane concentration remained relatively steady at around 88 %, and ethane concentration initially started just below 3 % and gradually rose to just below 4 %. An important indicator of gas origin is the ratio of ethane to methane. The floor seams have very low ethane concentrations, whilst the major roof strata reservoirs (Bulgo and Scarborough sandstones) have been estimated to have 2.9 % and 7.5 % respectively (dotted lines on graph).

Figure 13 shows that the ethane to methane ratio for the measured period was initially 3 % and over the measured period rose to just over 4 %. This clearly suggests that a major component of the extracted gas originated in the roof strata. There is, however a range of possible component contributions from the roof strata and floor seams which could generate this ethane to methane ratio. For instance, this ratio results from mixing approximately 50 % floor seam gas with 50% Scarborough gas with no Bulgo gas, and also from a mix containing mostly Bulgo gas with smaller amounts of Scarborough and floor seam gas. Issues with CO$_2$ coming out of solution from groundwater in the roof strata preclude using CO$_2$ as an indicator of the seam gas component. Investigations are currently underway to determine if more elaborate fingerprinting techniques might provide a better understanding on the component contributions from the individual roof strata and floor seams.

**Tailgate Gas Composition Monitoring**

During the operating period of WCC SGW No1, a number of ventilation and goaf gas samples were collected and analysed. Interpretation of the composition results calculated air free is shown in Figure 14, which indicates two distinct groupings (back of goaf samples and others), (Wood 2006). Ethane/methane results from the back of goaf areas are less than 0.01 while the other samples indicate a ratio of 0.01 to 0.04.
Based on ethane to methane ratios for Bulgo, Scarborough and floor seams of 2.9 %, 7.4 % and 1 % respectively, the gas samples taken from the tailgate corner of the goaf and outbye in the tailgate return had signatures consistent with greater than 40 % of strata gas in the methane fraction. A maximum of 72 % of strata gas was recorded from the tailgate corner. Back of goaf samples recorded a maximum of 30 % strata gas in the methane component of the mixture. The ratio of the source components in the tailgate return remain relatively constant through the range of methane concentrations measured (Wood, 2006).
APPIN COLLIERY SGW NO1 RESULTS

Appin SGW No1 was situated above Appin Longwall 408, approximately 620 metres outbye from the face installation road. The well was situated 40 metres from the tailgate. The surface location of the well was approximately 200 metres from a cluster of houses, necessitating the use of enclosed flares to dispose of the extracted gases. Refer Figure 4.

The longwall passed under the well on 6-1-06, resulting in a similar pressure response to that measured for WCC SGW No1 (see Figure 8). Delays in commissioning the goaf plant and enclosed flares, coinciding with consecutive record weekly longwall production rates, resulted in the plant not being started until the longwall was approximately 100 metres past the goaf well.

Upon commencement of gas extraction, it became obvious that the well flow rate would be constrained by the flare units to less than 400 lps. Flow rates above this level caused considerable lengths of flame to emanate from the flare stacks, and also led to the flare units overheating. The expectation was that the two flare units would have combined capacity for 600 lps, however extreme high purity extracted goaf gas (> 90 % CH4, with an additional 2.5 % higher order hydrocarbons) reduced the capacity to this lower level.

To determine the maximum flow capacity for the well, an unconstrained flow test was undertaken on 25-1-06, when the well was 170 m behind the longwall face. This test involved running the vacuum pump system at full capacity bypassing the flare units, and diverting the gas to a vent stack. The flow rates from this trial are shown in Figure 15. Prior to the commencement of the test, the well was shut-in. The shut-in well pressure was recorded as 0 kPa. The initial peak flow rate of approximately 750 lps NTP quickly dropped to a sustained value of around 420 lps NTP with a -50 kPa suction pressure applied to the well. Interestingly, this sustained rate is only marginally higher than the demonstrated capacity of the enclosed flares.

![Fig. 15 - Appin SGW No 1 Unconstrained flow test on 25-1-06](image)

At no stage during its operation did the Appin SGW No 1 well show any influence on the tailgate gas levels. In fact, gas levels measured underground at the time were significantly lower than expected, indicating an unusual reduction in the specific gas emission (SGE) for longwall extraction. Not surprisingly, this low SGE condition coincided with a record production month for the mine. It is likely that this low SGE was in part due to an extensive and sustained campaign of draining gas and fluid from the Bulgo Sandstone from a network of 6 in free flowing holes.
WEST CLIFF COLLIERY SGW#2 RESULTS

Initial Connection

WCC SGW No2 was situated above West Cliff Longwall 31, approximately 1,450 metres outbye from the face installation road. The well was situated 40 metres from the tailgate drive. Refer Figure 3. The main difference between this well and WCC SGW No 1 was the slotted casing finish depth, which for this well was just 10 metres above the Bulli Seam, whereas WCC SGW No 1 had it finish 35 metres above the Bulli Seam.

The longwall progressed under SGW No 2 on 23/3/06. Evidence of connection between the goaf and the well was first seen on 20/3/06, with a significant suction pressure of -75 kPa generated at the wellhead, indicating the well water level was dropping. This continued through till 24/3/06 when wellhead pressure changed to -2 kPa suction. On 27/3/06 the goaf plant was turned on, with the longwall approximately 25 m past the well.

WCC SGW No 2 – Production History

A full plot of plant suction pressure, measured gas flow rate and face position relative to the well is shown in Figure 16. Upon plant start-up, initial flow of approximately 800 lps was achieved with the maximum suction pressure of 9 kPa. However, at this rate oxygen levels increased to over 5 % necessitating throttling back the plant to around 550 lps, achieved by reducing suction pressure to between 5-7 kPa. Entering the second week of operation the well flow rate dropped significantly to 350 lps. This indicated a substantial loss in connectivity between the well and the open goaf zone. At this stage the wall was 90 metres outbye of the well. As the longwall progressed further away, the well flow rate continued to drop reaching a low of around 200 lps. At this flow rate the oxygen levels consistently remained below 1 %.

Figure 17 is a plot of tailgate gas levels coinciding with the initial plant start-up at 11:00 am on 27/3/06, and the subsequent 18 hour period which include a plant shut-down due to generator problems. Upon plant start-up, tailgate gas levels dropped by 0.9 % in a 1 hour period. This reduction was maintained until the generator faulted at 1:30 am. Methane concentration gradually rose 0.9 % to original levels over a 6 hour period. At 9:00 am on 28/3/06 the plant was re-started, with a rapid reduction in tailgate gas back to the lows achieved the previous day.

Several free-vent and shut-in tests have been conducted on WCC SGW No 2. Typical free-vent flow rates of less than 50 lps have been recorded, with shut-in pressures of around 2 kPa measured at the wellhead. Subsequent to the first week’s operation, no measurable effect on tailgate gas levels was noted during well shut-in tests. Based
on these observations, it is likely that a severe restriction developed between the well and the open goaf area at the end of the first week’s operation. This would most likely be either recompaction or pinching of the casing due to ground movement associated with caving. The slightly positive pressure measured during the well shut-in tests is a result of the battle between the mines’ negative ventilation pressure and the positive pressure generated by the buoyancy of methane.

![Fig. 17 - WCC SGW No 2 Reduction in tailgate gas on plant start-up](image)

**WCC SGW No 2 Extracted Gas Composition History**

Regular bag samples were collected from WCC SGW No 2 gas stream since it commenced operation on 23/3/06. Figure 18 shows gas composition history from well start-up to time of preparation, calculated on an air-free basis. Similarly to WCC SGW No 1, methane was consistently around 90% with ethane ranging between 3-4%. Based on the ratio of these values, it is likely that a significant component of the extracted gas originated in the roof strata sandstone formations.

![Fig. 18 - WCC SGW No 2 extracted gas composition plot](image)
GOAF GAS RESERVOIR MODEL

In general, the effect of coal extraction on over and under-lying strata is initially to reduce the vertical stress, which typically results in failure of the roof and floor material due to high unconfined horizontal stresses. A resultant of this failure is the creation of vertical fractures which allow gas to flow from pressurised formations and seams into the goaf. Re-compaction theory has the vertical stresses rising to near original values as the longwall face progresses away from a particular location. Of importance is the extent to which recompaction might close down these vertical flow paths, thereby limiting or preventing further gas migration into the goaf.

Standard goaf compaction models declare that for competent roof material a zone of highly re-compacted, low permeability goaf is created in the central bulk of the goaf area, with lower compaction, higher permeability zones extending around the goaf fringes - behind the face and inbye adjacent to the gate roads. It is likely that vertical fractures above or below these higher compaction zones will seal up, whereas vertical fractures leading to the goaf fringes may stay open and provide gas migration pathways.

Obviously the above process is heavily influenced by the stratigraphic and geomechanical properties of the individual stratum, as well as operational factors including panel width and extraction rate. For instance, the Stanwell Park Claystones are noted for their highly plastic behaviour, and extreme low permeability. The amount of vertical fracturing induced in this formation by extraction, and the time that such fractures remain open is not known. What is known is that mining induced vertical fractures through this material are necessary for the overlying strata gas to reach the goaf.

Another significant feature of standard goafing and subsidence models is dilation occurring between bedding planes, creating horizontal gas flow paths. It is possible that these dilations play a major role in gas reaching any open vertical fractures, probably concentrated around the goaf fringes.

Figure 19 is a 2-D schematic representation of the goaf. It illustrates that the flow of gas into the goaf is pressure driven, and that the pressure differential between interburden strata layers is dependent not only on the permeability of the strata, but also on the extent and dilation of mining induced fractures. The flow of gas is dominated by joints, fractures and other highly permeable flow paths. A limiting factor to flow rate is the low matrix permeability of the host rock or seam, through which the gas must migrate before it can enter the more permeable flow paths. This is obviously influenced by the degree of mining induced fractures.

As previously discussed, recompaction will cause closure of fractures, but the goaf fringes undergo less re-compaction than the centre of the goaf. Horizontal dilation along bedding planes will assist the migration of gas towards the fringes where the gas can then flow through open vertical fractures. The driving pressure for roof gases is between 3-4 MPa, whilst the driving pressure for floor gases is up to 6 MPa. Particularly in the case for the floor gas, this pressure is sufficient to fracture interburden if unconstrained vertically.

If the re-compaction model is correct, with likely closure of vertical fractures within the claystones and shales as vertical stress increases, then the close match between the inferred “gas-in-place” estimate in Table 1 (45 m$^3$/t) and the measured specific gas emissions reported for Appin Longwall 402-405 (35-40 m$^3$/t) imply that the horizontal dilations play an important role in gas migration into the goaf. Without these horizontal flow paths, it is likely that less gas would reach the goaf and specific gas emissions would be less. That is, re-compaction would seal flow paths before all the potential available gas had migrated into the goaf. From Table 1, the potential specific gas emission from roof strata is 15 m$^3$/t of coal mined, with approximately 50% of this gas coming from the Bulgo Sandstone. For Bulgo gas to reach the goaf it must pass through the extremely low permeability and relatively plastic Stanwell Park Claystone formation and then further down, the Wombarra Shale formation. Obviously, this process is reliant on a network of vertical fractures being formed that extend upwards through these low permeability zones.

Evidence for the role that horizontal dilations provide to gas migration can be seen from the free venting characteristics of WCC SGW No 1, discussed in Section 7.2. For the free-venting period of 10-3-06 to 1-5-06, a sustained flow averaging 320 lps was achieved. During this period, well shut-in pressures of 75 kPa were often measured. These relatively high shut-in pressures indicate that the flow is pressure driven, not buoyancy driven. Gas composition analysis confirms the gas predominately originates in the roof strata. A probable conclusion is that horizontal dilations must be acting as conduits for this gas to migrate from the source rock towards the well.
The behaviour of the various goaf wells during shut-in tests provide some insight into the characteristics of the goaf and caved zone in terms of the pressure distribution and permeability. WCC SGW No 1 typically reached a wellhead pressure of around 75 kPa within 30 minutes of shut-in. WCC SGW No 2 and Appin SGW No1 only ever reached a shut-in pressure of 2 kPa, which is likely generated by buoyancy. The difference can most likely be explained by the fact that the casing of WCC SGW No 1 well finished 35 metres above the Bulli Seam in the Wombarra Shale, whereas for WCC SGW No 2 and Appin SGW No 1 the slotted casing sections finished just 10 metres above the Bulli Seam in the sandstone roof. This indicates that the additional 25 metres of roof material between the bottom of WCC SGW No 1 and the extracted seam was of sufficiently low permeability to generate this 75 kPa pressure.

Another significant observation occurred at the Appin SGW No 1 during plug and abandonment (P&A) procedures (slotted casing finished 10 metres above Bulli seam). Prior to the P&A commencing, the well was observed to suck in air when open, indicating good connection to the goaf and mine ventilation circuit. The first component of the P&A involved filling the slotted casing interval with sand and placing a small cement plug on top of the sand. After placement, the wellhead pressure was observed to rise to 650 kPa overnight, this pressure most likely coming from Bulgo gas flowing through a breach in the 10 in non-slotted casing. Prior to P&A, the casing was conducting this gas to the goaf. After filling the slotted casing with sand and the cement plug, this flow path was eliminated, explaining the observed pressure build-up.

In general, the casing designs for the goaf wells (slotted to just below the base of Bulgo Sandstone) result in the extracted gas coming from high in the goaf area. It is therefore not surprising that the majority of the extracted gas is from the roof strata, as evidenced by the ethane-methane ratio of collected samples. In the case of the WCC goaf wells, analysis of tailgate gas composition indicated that a significant proportion of strata gas (>40 %) was also reporting to the tailgate.
CONCLUSIONS

A Surface Goaf Well Trial Program has been undertaken to determine the effectiveness of this technique to reduce gas concentrations within the longwall ventilation circuit, and minimise gas related production delays. Three trial wells have been completed with small variations in the depths at which the goaf wells were terminated above the Bulli Seam.

Considerable production variation between the three wells was recorded, although no conclusive causes for this have been identified. All three wells produced predominately strata gases, as identified by fingerprinting using ethane-methane ratios. The two West Cliff wells were observed to have significant effect on gas concentration levels in the longwall ventilation circuit. The Appin trial well had no noticeable effect on longwall gas levels.

In consideration of the general behaviour of the three wells, it is proposed that the well flow rates and influence on longwall gas concentrations are due to a complex interaction between geological and geomechanical factors. High permeability flow paths are created in mining induced vertical and horizontal fracture systems. These systems tend to close as the longwall moves away and recompaction occurs, although this effect is reduced towards the goaf fringes. Further work is required to develop a better understanding on the 3-D properties of the goaf in terms of permeability variations and pressure distributions.

A more detailed analysis of the effect the goaf wells have had on longwall gas levels is required to fully evaluate the benefit provided by the wells in terms of improving longwall production.

REFERENCES

Wood, J. 2006. Assessment of the influence of West Cliff Surface Goaf Well 1 on Underground Gas Conditions, Internal Report
DEVELOPMENT OF IN-HOUSE COAL SEAM PERMEABILITY TESTING CAPABILITIES

Tim Cummins¹ and Luke Fredericks¹

ABSTRACT: Coal seam gas has historically been and still remains a major issue for underground Illawarra Coal’s mining operations in the Southern Coalfield. The control of seam and strata gas is essential to maintaining operational safety and mining continuity. The Resource and Exploration department surface exploration drilling program is currently investigating coal seam gas characteristics through the use of coal permeability testing (injection fall-off testing). The need for coal permeability testing is essential for assessing the reservoir gas characteristics in the seams. Coal seam structure and reservoir properties are closely related to regional and localised geological structure, which greatly influence in situ coal seam permeability and reservoir characteristics.

Testing is conducted on all seams within the mining stress relaxation envelope of the mined seam. Specialist software (PAN Systems) is used to process the data and interpretation is undertaken in conjunction with all other available data-sets.

Permeability testing is critical for understanding the gas regimes within Illawarra Coal’s four mining lease areas. Permeability data is needed for the Bulli through to the Tongarra coal seams primarily for mine gas drainage, evaluation of Coal Bed Methane production potential of deeper seams in the Illawarra Coal Measures and identification of zones of variable gas drainability and outburst potential.

INTRODUCTION

Coal seam gas has historically been and still remains a major issue for underground Illawarra Coal mining operations in the Southern Coalfield. The control of seam and strata gas is essential to maintaining operational safety and mining continuity. The Resource and Exploration department surface exploration drilling program is currently investigating coal seam gas characteristics through the use of coal permeability testing (injection fall-off testing). The need for coal permeability testing is essential for assessing the reservoir gas characteristics in the Illawarra Coal measures.

There are a number of techniques for testing coal seams to determine its reservoir characteristics. These include, Injection Falloff, Drill Stem Test (DST) and Step Rate. To limit the standby time of the drill rig, only injection falloff tests are conducted. This involves isolating the individual coal seams and injecting fresh water at a constant rate to pressures below the predicted coal seam fracture point. The injection is then stopped and the hole sealed to allow the injected water to dissipate into the test interval. The injection phase usually takes 2 hours and the fall off 2-5 hours depending on the permeability, the lower the permeability the longer the pressure take to dissipate into the seam.

Illawarra Coal has conducts internal reporting and interpretations using the PAN System software with assistance from BHP Billiton Petroleum / Coal Bed Methane division.

REGIONAL GEOLOGY AND MINING OPERATIONS

Illawarra Coal has four operational coal mines including the Dendrobium Colliery in the south and West Cliff, Appin and Douglas Collieries in the north (Figure 1). The regional geology of the area is the Permian -Triassic sequences of the Sydney Basin. The stratigraphy of area consists of the Permian Illawarra Coal measures which are overlain by the Triassic Wianamatta and Narrabeen Groups (Figure 2).

Four major coal horizons within the Illawarra Coal measures are tested for coal seam permeability - the Bulli, Balgownie, Wongawilli and Tongarra seams. The Bulli seam is the mined seam for mines in the northern leases,
while the Wongawilli seam is extracted by the Dendrobium mine. Longwall mining operations within each mining lease area are bounded by geological features such as faulting and intrusions. Coal permeability is used to better understand coal seam reservoir characteristics; these are directly related to geological features in the immediate area. Permeability testing is then combined with other geological data-sets for developing and planning future mine plan orientations and coal extraction zones.
COAL SEAM GAS FLOWS

Conducting permeability testing and interpretation of results is vital for understanding coal seam characteristics and gas flow regimes within Illawarra Coal Holdings. Gas flows from coal seams over and underlying the working economic seams flow into the goaf area after longwall mining. This effect increases the gas regime, and has the potential to “gas out” the mine. This significantly impacts on mining operations as mining continuity is disrupted and coal extraction cannot resume until the gas is drained to below safe working levels. Coal seams that contain high gas contents have the potential to outburst and injure workers. These zones must be drained to a suitable level before any mining can occur. Early identification of low permeability coals allows larger lead times for pre-drainage from underground or the surface if a long drainage time is required. The data obtained from permeability testing is incorporated in the planning of systematic gas drainage which can be implemented before the commencement of mining.

<table>
<thead>
<tr>
<th>AGE GROUP</th>
<th>SUB-GRP</th>
<th>CODE</th>
<th>FORMATION &amp; MEMBERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>WIANAMATTA</td>
<td></td>
<td>WMSH</td>
<td>BRINGELLY SHALE MINCHINBURY SANDSTONE ASHFIELD SHALE</td>
</tr>
<tr>
<td>NARRABEEN</td>
<td></td>
<td>HGSS</td>
<td>MITTAGONG FORMATION</td>
</tr>
<tr>
<td>SYDNEY</td>
<td></td>
<td>CHSM</td>
<td>ECKERSLEY FORMATION</td>
</tr>
<tr>
<td>CUMBERLAND</td>
<td></td>
<td>PMSH</td>
<td>PHEASANTS NEST FORMATION</td>
</tr>
<tr>
<td>SHOALHAVEN</td>
<td></td>
<td>BMSS</td>
<td>Broughton formation BERRY SILSTONE NOWRA SANDSTONE WANDRA SANDSTONE WANDRA SANDSTONE SILSTONE SNAPPER POINT FORMATION</td>
</tr>
<tr>
<td>TALATERANG</td>
<td></td>
<td>ACM</td>
<td>CLYDE COAL MEASURES</td>
</tr>
</tbody>
</table>

Fig. 2 - Stratigraphy of the southern coalfield

TYPES OF WELL TESTS

There are a number of techniques that can be used for testing coal seams to determine its reservoir characteristics, including, Injection / Fall off, DST and Step-Rate. DST involves promoting flow from the seam by controlled removal of the hydrostatic head by inserting an air cushion in the rods. Step-rate tests are conducted on injection wells to determine the maximum injection rate possible without fracturing the coal reservoir (Singh, and Krase, 1987). Step-rate testing consists of a series of constant rate injection periods, with rates increasing from low to high injection rates in a step wise fashion where each step period is of equal length of time.
Coal Permeability Injection / Fall Off Testing

BHP Billiton Illawarra Coal has been conducting in-house coal permeability testing since October 2004. Prior to this around 40 tests had been conducted and in the past year and a half an additional 130 tests have been done. Testing procedures have since been constantly refined, documented and updated allowing more reliable results to be produced, and less coal seams being fractured during well testing.

Injection / fall off testing involves injecting fluid into the reservoir to increase bottom hole pressure, and a subsequent shut-in period and pressure drop during the fall off period (Bourdet 2002). Permeability testing equipment lowered down the wire-line can be simply and efficiently added on to existing drilling equipment which eliminates the need for a separate testing rig. A schematic of the double inflatable packer system used in borehole testing equipment that is supplied to Illawarra Coal by AGE Developments is shown in Figure 3.

Permeability testing of Coal seams is conducted during the drilling of an exploration borehole which minimises stand by times and the duration of drilling the hole. Permeability tests are conducted on coal seams based on their potential gas content and contribution to the goaf, and reservoir thickness which is usually greater than 0.7 cm.

![Fig. 3 - Equipment used for down-hole coal seam permeability testing (AGE Developments)](image-url)
WELL TESTING PROCEDURES

Injection testing is conducted once the coal seam and a suitable sump (that includes at least 50 cm of the coal floor) has been drilled and recovered. The coal seam is drilled using fresh water to reduce the influence the drilling fluids, which have the potential to reduce permeability (Figure 4). The hole is first flushed with fresh water to further remove drilling fluids, particularly where mud was required to weight the hole. Drilling mud infiltrates the walls of the borehole increasing the skin effect, which will be discussed in the interpretation section. Appropriate amounts of drill rods are either added or removed so that the bottom of the packer will be set in the roof of the coal seam so that the test interval is the coal seam horizon.

Two dual recording down-hole memory gauges that record seam pressure and temperature at two second intervals are attached to the bottom of the inflatable packer. An air line that which inflates the packer system with nitrogen gas is also attached to the packer. The packer is then attached to the drill rig’s wire-line and run-in to the bottom of the hole to the bit seating sub on the diamond drill bit. The packer is then inflated at a nominal pressure 2.76 MPa (400 psi) over the hydrostatic pressure head of the borehole in order to overcome packer deflation. Hydrostatic head pressure in fresh water is determined by the following formula.

\[
\text{Hydrostatic Head Pressure} = \text{Depth of Coal} \times 1 \text{ meter of head of kilopascal (9.80)} \times 0.145 \text{.} 0.145 \text{ is the conversion factor to pound-force/square inch (North).}
\]

Once fully inflated, the top packer seals the rods and the bottom packer which is seated below the drill bit seals on the borehole wall. The two down-hole packers are designed to seal the testing section off from the annulus. The rods are filled with fresh water removing any gas. The top of the drill rods is fully sealed at the surface with pack off seals that prevents water leakage.

A surface pump with hydraulic rate controls is used to inject water into the borehole via hose lines and flow gauges. This can inject water at a constant rate, pumping as little as 0.10 to 600 litres per hour, up to a pressure of 4.14 MPa (600 psi). The injection delivery rate from the pump is kept constant at the surface during testing once the down-hole pressure begins to rise.

Test data is recorded at the surface with a flow meter manifold that consists of a flow meter and a pressure gauge. The analogue pressure gauge displays down-hole pressure in the test interval to a maximum pressure of 4.14 MPa (600 psi) and the analogue flow meter records water volume pumped into the hole from the injection pump. A digital flow meter has recently been used to improve accuracy. The flow meter is connected via hose lines between the pump and the analogue flow meter and accurately records injection rates as small as 0.10 litres per minute to a maximum of 9.5 litres per minute and cumulative volumes injected down-hole.

**Injection Phase**

Testing involves isolating the individual coal seams and injecting fresh water at a constant rate to pressures below the predicted coal seam fracture point. This fracture point is calculated from the estimated Closure pressure (Pc) of the coal seam using known values of the Poisson’s ratio, overburden and pore pressures gradients in the borehole using the formula.

\[
Pc \text{ (estimate)} = \left(\frac{v}{(1-v)}\right)\text{Overburden Pressure} - \text{Pore Pressure}
\]

Poisson’s Ratio = v, assuming v = 0.25 for Coal
Overburden Pressure = 2.5 SG (1.08 psi/foot)
Pore Pressure = 0.433 psi/foot.

Therefore: \(Pc = 0.33\times(1.08 -0.433) +0.433Pc = 14.6 \text{ KPa/meter (formula supplied by CBM group, Don McMillon pers comms).}\)
The Injection phase is stopped before the estimated closure pressure is exceeded, or after a nominal period of 2 hours testing. As coal seam and interburden depth varies between exploration holes in the four mining areas and generally increases towards the north, values of closure pressure also increase. Values range from a minimum of 1.7 MPa (250 psi) in the Dendrobium area for the Wongawilli seam, up to a maximum of 3.3 MPa (480 psi) in the Tongarra seam in the Douglas Area. During the testing phase the injection rate is recorded. The rate may be altered due to the hole state during the test, surface gauges give an indication of bottom hole conditions.

**Fall-off Phase**

After injection has ceased, the fall off period begins where pressure is allowed to dissipate into the test interval and down-hole pressure re-equilibrates to coal seam reservoir pressure. This period generally takes 2-5 hours depending on the permeability of the seam. The lower the permeability the longer the pressure takes to equilibrate to reservoir levels. The fall off phase ends when the bottom-hole pressure increases, this increase in pressure is called the end point of the phase. The air line is bled off and the packer system is deflated and retrieved at the surface as the wire-line is wound up.

Contributing factors that were identified in well tests for early termination of the fall off phase included drill rods, packer, air line leakage and under pressured seams. Leaks in the system relating to testing equipment or coal seam properties result in pressure being dissipated back into the borehole. This pressure decrease can be accounted by leaking or loose fitting hose-line and air line connectors and/or gas leaks through the packer threads or the air line hose. In under pressurised coal reservoirs, the pressure is less than the hydrostatic head and during the fall off period, the down-hole pressure in the reservoir is reduced. This reduction causes a vacuum effect where water is drawn down from the hydrostatic head and is injected into the reservoir, thus raising the down-hole pressure and effectively ending the fall off phase.

**IN-HOUSE REPORTING AND DATA COLLECTION**

Illawarra Coal has identified the need to for internal interpretation and reporting of permeability test results. In-house interpretations done by Illawarra Coal allow for efficient, and cheaper modelling of data that is completed in a timely fashion as opposed to external reporting of results by outside consultancy firms. Data interpretations and modelling on a single well can be made immediately on data retrieval from the gauges.
Data Interpretation and Results

After initial collection the data from the two down-hole memory gauges are loaded into the PAN System software. Hole properties are then entered into PAN System

- Predicted seam pressures and temperature
- Hole diameter
- Test interval and wellbore storage coefficient, calculated from the volume of fluid in the test interval including the drill string.

Injection rates are assigned to the pressure time curve (Figure 5). Note the rate change at the start of the injection period.

![Test Overview](image)

**Fig. 5 - Injection fall off graph of raw pressure data Vs time with the injection rates**

Both the injection and fall off periods are interpreted independently and then the entire test is interpreted. The wellbore storage value is calculated from a single slope line, and the radial flow is selected using a zero slope line from a log-log graph of dimensionless time and pressure (Figure 6). These values are refined by the semi-log plot where the straight lines can be manipulated to better fit the pressure curve. Further refinement by fitting a type curve (Figure 7) which gives a variety of idealised curves with varying permeability and wellbore storage from the previously entered parameters.

The results are the run through computer generated matches, quick-match, to refine the interpretation. The final process involves a fully automatic computer generated match using the Auto-match function, Figure 7. This match involves selecting the injection and falloff data and injection rates and using the well parameters initially entered into the program to calculate values for the test. Numerous parameters are obtained from the test such as; permeability, wellbore storage, skin, temperature and the radius of the hole tested.

Complications during the interpretation result from numerous factors during the test period. Leaks in the system can occur through the rods, out of the headworks and past the packers. Leaks must be eliminated or false interpretations can result. Test induced changes to the test horizon also have a detrimental effect on the interpretation. Induced fractures during the injection period effectively restart the test period as down-hole conditions have been changed.
**Fig. 6** - Log-log plot of dimensionless time and pressure

**Fig. 7** - Test Result overview using quick match results
Fractures result when the critical seam fracturing pressure is exceeded during the injection phase. This pressure is estimated prior to the test, however, some seams are under-pressure, standing water level over 200 m below the potential hydrostatic head. This results in the fracture pressure being reduced by over 280psi. After observation of this phenomenon a falling head test is conducted to estimate the actual value.

**PERMEABILITY MODELLING**

Permeability testing is critical for understanding gas regimes for Illawarra Coal’s four mining leases. Permeability data is required for:

- Mine gas drainage planning
- Evaluation of Coal Bed Methane production potential of deeper seams in the Illawarra Coal Measures.
- Identifying tight coal zones, structures, coal fracturing
- Mine planning

**Mine Gas drainage and mine planning**

Permeability testing conducted on a regional scale over a mining lease area can provide a general flow regime of gas flow in the area and help manage and reduce the potential of outburst. Areas of poor drainage, and/or with or high CO₂ or CH₄ gas contents are identified, and drained well before coal extraction occurs. Permeability testing is used in conjunction with surface and underground geological mapping, and gas contouring for medium term mine planning.

**Coal Bed Methane Production**

Collection and analysis of well test data is vital in accurately estimating the in-situ natural fracture system permeability (Mavor and Saulsberry). Permeability testing can identify areas of over pressured and under saturated coal reservoirs, as well as identify areas of high potential for methane production.

**Coal Permeability Modelling**

Permeability measured in Millidarcy’s (md), is the ability of a fluid to pass through a porous medium (Mavor and Saulsberry). Permeability values across Illawarra Coal mining areas are highly variable due to geological conditions in the coal seam, and different structural domains between different mining lease areas. Average permeability values for coal seams across Illawarra Coal mine lease areas are shown in Table 1.

**Coal Seam Permeability**

<table>
<thead>
<tr>
<th>Seam</th>
<th>Mining Lease</th>
<th>Appin</th>
<th>West Cliff</th>
<th>Douglas</th>
<th>Dendrobium</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulli</td>
<td></td>
<td>3.6</td>
<td>2.0</td>
<td>1.7</td>
<td>N/A</td>
</tr>
<tr>
<td>Balgownie</td>
<td></td>
<td>4.2</td>
<td>N/A</td>
<td>0.5</td>
<td>N/A</td>
</tr>
<tr>
<td>Wongawilli</td>
<td></td>
<td>0.7</td>
<td>0.3</td>
<td>2.6</td>
<td>1.6</td>
</tr>
<tr>
<td>Tongarra</td>
<td></td>
<td>0.5</td>
<td>N/A</td>
<td>0.1</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Coal permeability is modelled by Illawarra Coal Resource & Exploration using Surfer 8 which is a gridding, contouring, and surface mapping program. Permeability values in Millidarcy’s are geo-referenced and contoured in 2D using the krigging method. Figure 9, shows a typical contour map derived from surfer. The Petroleum / Coal Bed Methane Division incorporates permeability results with other mine planning and geological data sets into Petrel software which calculates drainage rate potential throughout specific mine areas taking in consideration all geological conditions. The resulting data allow drainage plans to be created indicating what type of drilling program is required and what lead time to use. Limitations of permeability testing arise from 700 m drill hole spacing between exploration holes and potential unknown or unconfirmed minor structures such as faulting and intrusions that both act as a barrier to gas drainage and may significantly increase or decrease
effective coal permeability. Individual test can be influenced greatly by localised geology. However the more tests conducted permit a region by region pattern to be developed.

![Simplified 2-D contouring of permeability results in the Appin area.](image)

**Fig. 8 - Simplified 2-D contouring of permeability results in the Appin area.**

**Coal seam geology and structural impacts on Injection well testing**

Structures play an important role in affecting local coal seam properties, coal permeability and the injection phase of well testing. Even the drilling process can affect coal seam properties with the vertical unloading of stress from retrieval of core can release stresses perpendicular to the drilling direction (North). Drill core that has been affected by faulting displays shattered coal, shear zone features, crumbling of core, as well as increased jointing and joint re-activation. Localised intrusions at coal seam level also increase jointing and fracturing within the coal seam.

Permeability testing can identify hard and soft coal zones that are not encountered with surface exploration boreholes. These zones are related to structural and intrusive features. Soft coal has a lower than expected fracture point due to the weaker structural properties of the seam. Interpretations on soft coal zones are difficult, though these zones have a good potential for gas drainability. Hard coal zones generally have lower permeability and a lower potential for lower gas drainability especially in CO$_2$ rich environments.

Cleating forms in response to local or regional folding and stress directions within the coal. Aquifers preferentially flow along the cleating and natural bedding features of the coal reservoir (Fetter 2001). Cleats within the coal can be forced open from well testing with increased down-hole fluid pressure, and flow can then be stimulated through them. Opening of cleat structures are observed in test results in a minor decrease in pressure, and a subsequent build up, skin values are negative on the Injection phase and positive on the fall off phase as they close with the decrease in down-hole pressure.

Under pressured coal seams contain less reservoir pressure than the hydrostatic pressure gradient of the borehole, hence the return flow rate to the surface is less than the rate it is injected in the hole from the delivery pump before the shut-in valve is operated. Over pressured seams contain a saturated coal matrix with a maximum amount of sobred gas under present reservoir conditions (Mavor and Saulsberry). The injection phase is generally shorter than for that of under pressured or general dual porosity reservoir systems.
CONCLUSIONS

BHP Billiton Illawarra Coal uses coal seam permeability testing for defining and assessing conditions in the coal reservoir system, and the effects of geological structure on the coal reservoir. Permeability testing is useful for defining regional gas flow regimes in order for maintaining continuity in underground mining operations, and for future mine planning purposes. The Resource & Exploration Department has developed and refined in-house procedures for permeability testing, and for interpretations on well test data using PAN system software. Interpretation of well tests using PAN software allows for a detailed and accurate analysis on coal seam reservoir properties, and responsiveness of a well to injection testing.

REFERENCES

MacMillon, D, 2005. Personal communication, 8 November.
A WEBSITE ON COAL AND GAS OUTBURST MANAGEMENT

Devendra Vyas¹, Najdat I Aziz¹, Richard Caladine¹, Lucia Tome¹

ABSTRACT: The University of Wollongong received funding from the Australian Coal Association Research Program (ACARP) in 2005 to develop a website on coal and gas outbursts in the Australian environment <www.uow.edu.au/outburst>. The primary objective of the website is to provide the coal mining industry with the information on outburst occurrence and mechanisms of outburst phenomenon and the means of treating it. The online system provides access to the experiences, knowledge and information acquired by the coal mining industry, research organisations and educational institutions in a quality controlled environment. Although the website is specific to the Australian scene, it nevertheless contains information which includes issues beyond Australian borders. The website to date is the culmination of the work of a team of professionals across the university, but not all are related to the mining profession. The information uploaded on the website includes reporting on the latest operational and research activities accumulated from field studies, as reported in various seminar presentations, conferences, and other publications.

INTRODUCTION

In February 2003, a workshop ACARP Outburst Research Needs, held at the University of Wollongong, identified a need for the establishment of a website on coal and gas outburst. This need was later reflected in the ACARP Sponsored Scoping Study Project (ACARP project C10012). One of the recommendations of the scoping study was the establishment of an online information management system in the form of a website on outbursts of gas and coal in underground coal mines. As a consequence ACARP funded a project (C 14015) to provide support for development of the outburst website developed at the University of Wollongong. The funding came into effect in May 2005 and a dedicated website developer was recruited for the project; the website is currently in the second year of development.

The primary objectives of this project are:

a) To develop a quality website for outbursts disseminating information, knowledge and experiences acquired by the Australian coal mining industry and research organisations.

b) To consolidate information that presently exists in conference proceedings, websites or as the experiences and knowledge of people working in the mining industry or in research.

c) To provide resources that have been filtered, selected, evaluated and organized for its primary audience: researchers, students and mining practitioners. Quality controls will ensure a selective and comprehensive collection of resources.

d) To provide a body of knowledge that is not only collated by the website, but also a reference point that will represent a ‘critical mass’ of information on this topic. Information that will be used to create and build new knowledge.

e) To provide practitioners with a discussion forum to openly communicate issues and concerns and network across geographical boundaries via a Chat Room.

PROJECT DEVELOPMENT

Initial construction of the website involved the expertise of a team of professional staff from across the University including a PhD student. Each team member has contributed in their area of expertise in terms of content, design and the legal aspects of the content being uploaded on the website. The website provides the coal mining industry with the necessary information on coal mine outburst phenomena. In addition it will also serve as a virtual forum for the exchange of ideas and information between mine operators, mining engineers, geologists, consultants and researchers in the field. The website is under construction with the aim of making it technically credible and to reflect the current status of outburst management and control, specifically on Australian conditions. Accordingly, the structure of the website falls into the following components:

¹ University of Wollongong
WEBSITE CONTENT

The technical content of the website comprises technical papers, ACARP reports, journals, presentations by mining industry personnel and an outburst scoping study, predominantly the work of Lama and Bodziony, (ACARP Project No C 4034, 1996). These items are further supplemented with technical material from industry personnel as well as specialist mining consultants. The reported case studies and the future ones to be incorporated will be supplied from mining personnel and expert industry consultants. Although the website is interlinked to various national and international websites it will not be used to actively promote any company, product or alike. The website’s primary function is to disseminate knowledge of the mining literature and share industry’s experiences. The ‘chat’ function would provide personnel with an opportunity to maintain networks within the industry, share knowledge and experiences and seek assistance from fellow peers.

The inclusion of a self-assessment component and feedback is vital to the capability and further development of the site. Positive feedback on the website has been received from the mining industry in various forums like ACARP meetings and outburst seminars.

The website is written in the standard web html format. Access to the site is via standard internet web browsers, for example, Netscape Navigator or Internet Explorer. Access the development site at http://www.uow.edu.au/outburst. A standard template has been developed and incorporated onto every web page to maintain consistency and ease of use for the user. The front page of the website can be viewed in Figure 1. The structure used for this website is as follows:

3.1 Universal Navigation System: This system allows the users to move around the web site with ease. The agreed system consists of a menu bar that is located in a column on the left-hand side of every page indicating a list of pages/modules that could be accessed from that particular page.

3.2 Content: The content of the web page takes up the remaining ‘real estate’. The content is presented in such a manner as to allow the user to read about a particular topic and then view a graphical diagram of that topic. The material incorporated into the web site is well researched and assistance is also sought from experts in the field.

When the user accesses the site the index page in figure above is displayed. A banner, “Mine Outbursts”, is incorporated at the top of the index page. Beneath the banner is a disclaimer and the site can be further accessed by agreeing to the terms in the disclaimer.

Why do we have the disclaimer? At the bottom is the “Enter” button which would allow entry to further contents of the website. The Content page starts with “Aim and Objectives” of the website. On the left hand side is the navigation bar indicating the contents of the website. On the bottom of the page a link has been provided to the “copyright format”. Any contribution to the website is welcome; however as per the “Australian copyright act 1968” copy right permission is required to upload the material on to the website. The copyright format can be downloaded and signed by the author whose material is to be uploaded. This format can be faxed or scanned and sent by e-mail to the Research Training Librarian, University of Wollongong.
The topics incorporated into this site are:

**Sitemap:** This module provides an overview of the contents of the website. Links to each and every topic in the website through the site map have been provided, except those areas which are still under construction.

**Definition:** This module is an introduction to outburst. It defines an outburst and provides links to related topics, Outburst size and Outburst cavern. The definition explains how an outburst occurs, the composition of an outburst and where it occurs. “Outburst Size” assists the user through definition and classification of outburst based on the size.

“Outburst Cavern” defines a cavern and the different types of cavern formed as a result of outburst along with photographs.

**Factors:** Factors contributing to the occurrence of an outburst is detailed under this section. The various factors are:

**Geological conditions:**
- Depth of mining
- Faults and folds
- Seam thickness
- Gas environment
- Gas content
- Mining induced stresses

**Coal properties:**
- Strength of coal seam
- Rank of coal seam
- Coal permeability
- Volumetric change
- Cleats and joints

**Management:** Mining of seams prone to outburst requires the development of specific procedures to ensure that the risk to miners and equipment is eliminated or reduced. The purpose of the management systems is to ensure that the procedures are in place and are precisely followed to ensure that the mining activities are done as per the management plan. Management of outburst includes:

**Prediction:**
2006 Coal Operators’ Conference

• Geology
• Prediction indices
• Monitoring
• Geophysical
• Seismic
• Electromagnetic
• Radar
• Radiometric
• Gas environment
• Gas type
• Gas pressure
• Gas content

Prevention:
• Ventilation
• Gas threshold value
• Gas drainage
• Pre-drainage
• Inseam drainage
• Post drainage
• Bore hole survey technique

Control:
• Ground de-stressing
• Gas drainage
• Bore hole survey technique
• Hydro facing
• Pulse infusion shot firing
• Blasting and borehole simulation
• Outburst hazard control
• Outburst management plan

Research and development: This section includes reports of ACARP on the subject as listed under. It include;
• ACARP gas and outburst workshop: 28th August 2004
• Gas and outburst workshop: 22nd November 2003
• Outburst scoping study- John Hanes
• Real time return gas monitoring for outburst and gas drainage assessment.
• Outburst scoping study- March 1996 (Lama & Bodziony)
• Outburst symposium- March 1995

Apart from this, various presentations which relate to the latest developments on mine outburst and the practices being followed at the mine sites at the outburst committee meetings, have also been uploaded and is an ongoing activity. The reports and the presentations can be downloaded from the website.

History: This module presents the historical facts of outburst incidence in Australia and worldwide. All efforts are being made to collect information from other countries as well. The Bulli seam outburst in Australia under “National and in Poland under “International” have been uploaded in this section.

Case studies: This section incorporates the case studies of Australian and international mines. Presently uploaded Australian case studies include the Central Colliery outburst, Queensland, which occurred in 2001, and those from Collinsville Colliery, Queensland in 1978, and Leichhardt Colliery, 1975 onwards.

Links: Links have been provided to other relevant national and international mining websites as a source of information on coal and gas outburst and has not been used to actively promote any company, product or alike.
FUTURE ISSUES

The issues to be addressed in the future are:

• Establishing a feedback form
• Chat rooms for two way communication and active discussions
• Uploading information on prediction, prevention and control of outburst
• Modifying the template to be inline with the latest trends.
• International history and case studies to be uploaded.

CONCLUSION

This website through information dissemination will provide:

• Increase awareness of outburst issues - practices and strategies
• Remote access to information
• Public awareness of issues related to outburst, coal mining and green gas effect
• Increase awareness of safety issues – prevention and management
• Availability of information in a timely manner

The website has been placed into the public domain to assist in the upgrading and training of the mining industry personnel as well as raising awareness of the mining operations to the public in general. The website is a valuable source and useful library for those interested bodies in remote regions and rural areas of Australia and also throughout the world. The website will represent a dynamic body of knowledge in this field. Information will be accessible in a virtual environment.

ACKNOWLEDGEMENT

The authors accord their appreciation to ACARP for having funded this project. Various mining companies, mining consultants, government organizations are providing material for the website. Significant material uploaded in the website in power point presentation are compiled and provided by John Hanes. The authors would also like to thank John Hanes and Adrian Hutton of school of geosciences, University of Wollongong, for their assistance with information, pictures and review of some of the material contained in the website. Thanks also extended to Illawarra gas outburst committee for their support.
BOLT SURFACE CONFIGURATIONS AND LOAD TRANSFER MECHANISM

N Aziz¹, H Jalaifar¹, ², J Concalves¹

ABSTRACT: A series of laboratory based push and pull tests were carried out to investigate how surface profile influence the load transfer mechanism of bolt/resin interface. Tests were carried out in both 75 mm and 150 mm long steel sleeves. Three types of bolts were examined, they were bolts most commonly used for strata reinforcements in underground coal mines in Australia. The bolts had near equal core diameter but of different profile configurations. The change in the length of the encapsulation sleeve was examined in light of the small number of profiles encapsulated effectively in short 75 mm long sleeves. The results showed that peak loads and displacements were directly related to the height and the spacing of the bolt surface profiles. Profile spacing appears to have greater influence on load transfer capacity than the profile height.

INTRODUCTION

Rock bolting plays an important role in ground support in both civil and mining engineering. Since it was first introduced, various studies have been undertaken to gain better knowledge about how rock bolts perform in different strata conditions. These studies have incorporated both the laboratory and field tests. In laboratory test, several methods of testing have been designed to evaluate the anchorage capacity of rock bolts. The conventional short encapsulation pull test involves pulling a bolt anchored in a hole either cast in concrete or drilled in rock. As an alternative, load transfers is examined using push and pull testing of short bolts in steel sleeves in a laboratory based environment. The laboratory load transfer test removes encapsulation problems encountered in the conventional short pull tests carried out in concrete blocks or in the field.

With the recent shift from mechanical point anchors to full encapsulation cement or chemical resin anchors, an area of attention is the bolt surface profile configuration as being a relevant parameter for load transfer mechanism interaction between the bolt and encapsulation medium. Fabjanczyk and Tarrant (1992) were the early researchers that recognised the importance of bolt surface configurations in influencing the load transfer mechanism interaction between resin and bolt interfaces. However, they made no reference on profile spacing. Aziz, Dey and Indraratna (2001) examined bolt profile configurations under constant normal stiffness conditions, indicating the importance of both bolt profile height and profile spacing as important parameters influencing the load transfer mechanisms. In their later work in short encapsulations tests, Aziz and Webb (a, b) examined the load transfer characteristics of both profiled and non-profiled bolts which established the role of profile spacing in load transfer capabilities. Their initial work was conducted by push testing of bolts in 75 mm steel sleeves with hole diameters being 27 mm holes. All the bolts used were equal core diameter of 21.7 mm in diameter. Aziz (2004), Aziz and Jalalifar (2005) carried out the tests under both push and pull test conditions, and that the bolts were conducted in a centrally located with uniform resin annulus thickness. Their work included the impact of resin encapsulation thickness variations, changes of bolt profile spacing, and the three dimensional modelling of both pull and push testing.

To address the limited length of the bolts encapsulated in 75 mm steel sleeve, an additional comparative study has been undertaken using 150 mm encapsulation length, with the tests being carried out under both push and pull conditions. The details of this study form the subject of discussion in this paper, together with a limited reporting of modelling analysis of the study.

LOAD TRANSFER CAPACITY

Load is transferred from the bolt to the rock via the grout by the mechanical interlock between the surface irregularities in the interface and friction. When shearing, the load is transferred to the bolt via shear stress in the grout. The nature of bolt failure in field test is different from laboratory test. In field test, failure is dependent upon the characteristics of the system and the material properties of individual elements. Slippage may occur at

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either of rock/grout or grout/bolt interfaces, which is called decoupling behaviour. Decoupling take place when the shear stress exceeds the strength of the interface strength. However, in the laboratory test, failure usually occurs along the bolt/grout interface. However, if real rock or concrete is used, instead of steel tube as outer casing element, then failure may happen along the rock/grout interface, depending on the strength of rock/concrete strength and hole wall profiling. Kilic A. (1999, 2002) reported that when surface friction of a borehole decrease, slippage occurs at the grout/rock interface.

In addition, when the borehole and bolt length exceeds a critical value, failure takes place at the bolt. Basically, the mechanical interlocking occurs when the irregularities move relative to each other. Surface interlock will transfer shear forces from one element to another. When the shear forces exceed the ultimate capacity of the medium, failure occurs and only frictional and interlocking resistance will control the load transfer characteristics of the bolt.

EXPERIMENTAL STUDY

Pull and push tests were carried out in two short encapsulation, 75 mm, and 150 mm length steel sleeves. Each bolt was encapsulated in the sleeve using Mix and Pour resin. As can be seen in Figure 1a the bolts were located centrally with uniform resin annulus thickness, and every effort was made to ensure the bolts were also set axially parallel to the sleeve hole axis. Figure 1b shows the general view of push test set-up in 150 mm cylinders. Because of the limited encapsulated length in 75 mm sleeve, there was insufficient number of bolt profiles embedded in resin encapsulation column, particularly for Bolt Type T3 with wider profile spacing of 25 mm. Accordingly the length of the steel sleeve was doubled by having two 75 mm selves butted at ends to form 150 mm long sleeve.

Figure 2a shows the laboratory set-up for pull test, in 150 mm cylinder. Figure 2b shows the post-test samples with the bolts being pulled out of the steel sleeves in 75 mm long, 45 mm outer diameter and 27 mm inner diameter. All failures occurred along the bolt grout interface. The grout and bolt properties are illustrated in Table 1. Tables 2 and 3 show various bolt parameters and experimental results in 75 and 150 mm encapsulation length respectively. Figures 3 and 4 show the post-test sheared bolt pushed out of steel sleeve in both 150 mm and 75 mm sleeve respectively. Figures 5 –7 show the profile of shear load–shear displacement in pull and push test in 75 and 150 mm sleeve cylinder respectively. As can be observed from both Tables 2 and 3 and in Figures 5-7, Bolt Type T3 has both push and pull loads and shear resistance values significantly higher than the other Bolt Types T1 and T2. This result is in line with previous results reported by Aziz and Jalalifar (2005). Post peak residual shear load and shear strength of the Bolt Type T3 was also higher than the other two bolts.
Fig. 2 - (a) Set up for pull test (b) Failure along the bolt grout interface in pull test

Table 1 - Grout and steel properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Grout</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>71</td>
<td>-</td>
</tr>
<tr>
<td>Ave. Shear strength (MPa)</td>
<td>16.2</td>
<td>645</td>
</tr>
<tr>
<td>E (GPa)</td>
<td>12</td>
<td>200</td>
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<tr>
<td>Poisson ratio</td>
<td>0.25</td>
<td>0.3</td>
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</tbody>
</table>

Fig. 3 - Failure along the bolt grout interface in push test at 150 mm encapsulation length

Fig. 4 - Post-test sheared bolt out of steel cylinder in 75 mm encapsulation length in push test
### Table 2 - The laboratory results in 75 mm encapsulation length

<table>
<thead>
<tr>
<th>Measured parameters</th>
<th>Pull</th>
<th>Push</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bolt type</td>
<td>Bolt type</td>
</tr>
<tr>
<td></td>
<td>T1</td>
<td>T2</td>
</tr>
<tr>
<td>Ave Profile Height (mm)</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>Ave Profile Spacing (mm)</td>
<td>11.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Ave Max Load (kN)</td>
<td>114.8</td>
<td>131.7</td>
</tr>
<tr>
<td>Ave Max Displ (mm)</td>
<td>4.10</td>
<td>4.51</td>
</tr>
<tr>
<td>Ave Shear Stress Capacity (MPa)</td>
<td>22.2</td>
<td>25.4</td>
</tr>
</tbody>
</table>

![Shear load as a function of displacement in pull test, 75 mm](image1.png)

### Table 3 - The laboratory results in 150 mm encapsulation length

<table>
<thead>
<tr>
<th>Measured parameters</th>
<th>Pull</th>
<th>Push</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bolt type</td>
<td>Bolt type</td>
</tr>
<tr>
<td></td>
<td>T1</td>
<td>T2</td>
</tr>
<tr>
<td>Ave Profile Height (mm)</td>
<td>1</td>
<td>1.35</td>
</tr>
<tr>
<td>Ave Profile Spacing (mm)</td>
<td>11.0</td>
<td>12</td>
</tr>
<tr>
<td>Ave Max Load (kN)</td>
<td>132.5</td>
<td>200</td>
</tr>
<tr>
<td>Ave Max Displacement (mm)</td>
<td>4.26</td>
<td>5.3</td>
</tr>
<tr>
<td>Ave Shear Stress Capacity (MPa)</td>
<td>12.78</td>
<td>19.5</td>
</tr>
</tbody>
</table>

![Shear load as a function of displacement in push test, 75 mm](image2.png)
Table 4 - Difference load between pull and push tests at both 75 mm and 150 mm sleeves

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Push load increase from 75 mm to 150 mm sleeve length (%)</th>
<th>Pull load increase from 75 mm to 150 mm (%)</th>
<th>Difference load between push and pull test at 75 mm (%)</th>
<th>Difference load between push and pull test at 150 mm (%)</th>
<th>Stiffness in 50 kN (kN/mm)</th>
<th>Stiffness in 100 kN (kN/mm)</th>
<th>Stiffness in first yield point (kN/mm)</th>
<th>Stiffness in peak point (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>11</td>
<td>15.4</td>
<td>11</td>
<td>7.6</td>
<td>36</td>
<td>34.4</td>
<td>-</td>
<td>30</td>
</tr>
<tr>
<td>T2</td>
<td>59</td>
<td>51.9</td>
<td>5.7</td>
<td>11</td>
<td>55</td>
<td>63</td>
<td>53</td>
<td>37</td>
</tr>
<tr>
<td>T3</td>
<td>63.6</td>
<td>58.2</td>
<td>7</td>
<td>10</td>
<td>40</td>
<td>39.9</td>
<td>28</td>
<td>17*</td>
</tr>
</tbody>
</table>

- This stiffness was calculated beyond the bolt shank yield point, so it cannot be measured as bolt stiffness.

![Pull test](image)

Fig. 7 - Shear load as a function of displacement in (a) pull test, (b) push test, 150 mm
Table 4 shows the difference value between the results of pull and push test in 75 and 150 mm sleeves. As can be seen from the Table 4, there is significant increase in shear load when 150 mm sleeve was used in Bolt Types T2 and T3, but not so significant in Bolt Type T1. The average difference between push and pull test, for all three Bolt Types, T1, T2 and T3 was between 7 to 11 % as deduced from Figure 8. And also it shows the shear stiffness in Bolt Type T2 is higher than other types of bolts. However, from the shear load-shear displacement trend it was found that the residual strength of the Bolt Type T3 is around 70 % of the maximum shear load in Bolt Type T2, which shows the significant effect of bolt profile spacing.

A comparison of the shear load for both pull and push tests are shown in Figure 8. Also the average shear stress values in long sleeve push test on each bolt showed a reduction of 44.2 %, 20.2 % and 18.2 % in Bolt Type T1, T2 and T3 respectively. Clearly the level of reduction in push load in Bolt Type T1 was more significant than the other two bolts.

![Fig. 8 - Shear load as a function of displacement in (a) push test, (b) pull test, 150 mm](image)

This was also true for pull test in which shear stress was reduced by 42.4 % for Bolt Type T1, 23.2 for Bolt Type T2 and 21.2 % for Bolt Type T3. The reason for higher load difference in 150 mm sleeve between push and pull in Bolt Type T3 was due to fact that the shear load was greater than the steel elastic yield load of around 260 kN. This caused relatively large bolt diameter reduction and the possible loss of bond connection, eventually dropping significantly in a shear load. The effect of excessive bolt elongation and diameter loss yield would be significantly reduced with the steel sleeve length being reduced to 120 mm in length, particularly for Bolt Type T3. However, the anchorage length for Bolt Type T1 can be exceeded up to 300 mm. The peak load/shear displacement in each bolt type was different, with Bolt Type T3 was significantly higher than the other two short spaced profiled bolts. Clearly, the profile spacing appears to have a relative greater impact on the load transfer mechanism of the bolt/resin interaction than the profiles height has. This supports the earlier findings under constant normal stiffness conditions reported by Aziz (2002). Also, increased profile pacing causes greater peak load - displacement, this is advantageous as it facilitates greater rock displacement and hence improved ground control capability particularly in soft rock conditions. The mechanisms of bonding between bolt, resin and rock can be attributed more on, friction and mechanical interlock than adhesion. Its worth mentioning that bolt necking occurs at the maximum ultimate applied load level at plastic yield stage, which for these bolts is between 330 and 350 kN pull load.

**NUMERICAL SIMULATION**

Next, the bolt, resin and interface behaviour was simulated by 3D numerical modelling using (ANSYS 3D).

The numerical simulation of the true bolt cross-section area and its ribs were found to be difficult, as it was almost impossible with the range of softwares available in the market today. However, a serious attempt was made to
model bolt profile configurations by taking into account the realistic behaviour of the rock-grout and grout-bolt interfaces, based on the laboratory observations. To achieve this task, the coordinates of all nodes for all the materials were firstly defined, then all these coordinates were inter-connected to form the elements and finally the elements were extruded, in several directions, to obtain the real shape. Finally, the numerical simulation was carried out for Bolt Type T1 in both pull and push test conditions. The relative simulation of Bolt Type T1, movement under pull test condition is shown in Figure 9. Two main fractures are produced as a result of shearing of the bolt from the resin. The first one begins at the top of the rib, with an angle of about 53 degrees running almost parallel to the rib orientation, and the second one has an angle of less than 40 degree from the bolt axis. At the fracture intersection, parts of the resin will chip away from the main resin body as it is overwhelmed by the rib surface roughness while shearing. The internal pressure produced by the bolt profile irregularities causes the tangential stress inducement in the grout. Grout fractures and shears when the induced stress exceeds the shearing strength of the grout material, thus allowing the bolt to slide easily along the sheared and slickenside fractures grout interface surfaces. The maximum bolt deflection occurs on the pulling side of the bolt, causing a reduction in bolt diameter.

![Fig. 9 - The bolt movement in pulling test](image)

Figure 10 shows the Von Mises stress trend along the bolt profile, which shows the maximum stress being concentrated at the pulling point of the bolt, gradually reducing towards its free end. Also it shows the shear and tensile stress trend along the bolt. The maximum tensile stress along the bolt is 330 MPa, which is almost equal to one half of the elastic yield point strength of 600 MPa. This means the bolt is unlikely to reach the yield situation and necking. Figure 10 also shows that there is low level of shear stress along the bolt.
CONCLUSION

Both the experimental and numerical results have lead to the following conclusions:

- The average shear stress capacity of a bolt in a push test is greater than in a pull test.
- Yielding and necking is unlikely to occur in bolts tested in 75 mm long steel sleeves as the peak shear load was around 40% of the maximum tensile strength of the steel. However, excessive bolt yielding and diameter reduction is likely to occur in 150 mm in Bolt Type T3 as the pull load is greater than the peak elastic yield load. Necking occurs at the maximum ultimate applied load level at plastic yield stage, which for these bolts is between 330 and 350 kN pull load.
- Bolt-resin interface failure occurred by initially shearing of the grout at the profile tip in contact with the resin. The load failure of the resin /bolt surface contact is dependent on the profile height as well as spacing.
- Increased profile pacing causes greater peak load - displacement, this is advantageous as it facilitates greater rock displacement and hence improved ground control capability particularly in soft rock conditions.
- Bolt Type T3 produced higher shear resistance, followed by Bolt Type T2 and then T1.
- Post peak load displacement profile of Bolt Type T3 is greater than the other Bolt Types T1 and T3 respectively.
- The length of steel sleeves used for load transfer mechanism study should facilitate a sufficient number of profiles encapsulation and in parity with the profile spacing of the bolt tests. Particular care must be taken to ensure the bolts installed centrally located in the sleeve and with the uniform resin annulus thickness.
- Bolt-resin interface failure occurred by initially shearing of the grout at the profile tip in contact with the resin.
- Numerical simulation provided an opportunity of better understanding of stresses and strains generated as a result of bolt resin interface shearing.
REFERENCES


A LABORATORY FACILITY TO STUDY THE BEHAVIOUR OF REINFORCED ELEMENTS SUBJECTED TO SHEAR

Luke Mahony¹ and Paul Hagan ¹

ABSTRACT: This paper outlines the design, construction, commissioning and subsequent investigation of a shear test laboratory facility at the University of New South Wales (UNSW). The facility was developed with the financial support of ACARP to understand the impact of shear loading conditions on the performance of reinforcement elements such as rockbolts and cablebolts. While it is acknowledged that shear loading commonly occurs, progress towards a better understanding of its impact has been limited due to issues in modelling behaviour. Preliminary testwork indicates that many of these issues have been overcome in the design of the current facility.

The facility is based on a single shear failure plane design which has overcome many of the issues associated with double shear designs such as dealing with high resultant loads and deformation to the support structure. A program of preliminary tests indicates that behaviour of the rock reinforcement system is less a function of the behaviour of the individual components than the interactions that occurs between the components that make up the system.

INTRODUCTION

Strata control is one of the core risk areas in underground coal mining. The use of appropriate technology for ground support whether it be roof or ribs, primary or secondary and the effective management of this technology can be pivotal to achieving a safe and economically viable mine.

Rock reinforcement elements provide a significant proportion of their ground control capacity through their ability to resist shear movement of the surrounding rock mass. This potential shear movement may take the form of sliding on horizontal bedding planes leading to strata bending; or block displacement along geological structures such as joints and other discontinuities. The shear resistance offered is far greater than simply the shear strength of the reinforcement element since the shearing action results in a normal or axial load in the element which is greatest at the shear plane and dissipates with distance away from the shear plane. The normal force clamps together the rock surfaces, mobilising the frictional properties acting over the large surface area of rock thereby contributing to frictional resistance.

While much research in the past has focused on the effects of axial loading of reinforcement elements in rock, less attention has been directed at understanding how these elements behave when subjected to shear loading. It is widely acknowledged that shear loads can have an important bearing on the stability of underground excavations; however, progress in this area has been hampered by the complexity of building a physical model that can reliably simulate shear loading conditions and the interplay that occurs between the different components.

The research conducted at the School of Mining Engineering at The University of New South Wales (UNSW) initially involved a review of the current understanding followed by design and building of a shear test laboratory facility to monitor performance of reinforcement elements and undertaking a preliminary round of experiments to begin to understand the behaviour of reinforcement elements on a rockmass.

The review found there is only a limited understanding. The review also highlighted the poor understanding of the effect of installation method of reinforcement elements and the influence of varying the loading conditions such as loading rate, pre-tension, torque and normal tension/loading on performance.

¹ School of Mining Engineering, University of New South Wales
DESIGN OF SHEAR TESTING FACILITY

The review as reported by Hartman and Hebblewhite (2003) commented that poorly designed and/or constructed test facilities had often had a detrimental impact on the quality of subsequent research activities in terms of integrity of results and/or validity of the findings. Hence a much greater effort in the design phase was required than had originally been envisaged with computer modelling of the load distribution being important in optimising the design.

The design objective was to be able to simulate conditions that are commonly encountered in underground mining environments. The initial design incorporated a double shearing action but it was subsequently found that a design with a single shear failure plane could be constructed with sufficient rigidity that minimal block rotation would occur about the reinforcement element. This design had an important advantage in that it effectively halved the load that needed to be applied to the system during testing and hence lowered the amount of reinforcement necessary to maintain stiffness of the facility.

In anticipation of the later experimental work, design of the laboratory facility had to cater for:

- determining the shear displacement along an anticipated plane of weakness and final deformation
- loading of the system at right angles to the shear load
- loading of the system at some acute angle to the shear load
- determining the axial load distribution along the reinforcing element as a result of the shear load and displacement
- determining the stress distribution around the reinforcing element (within the concrete/rock mass)

The as-built laboratory facility is based on a hydraulically actuated Avery-Denison compression test machine with an axial load capacity of 3600 kN. The facility has a rated shear loading capacity of 600 kN and is capable of isolating many of the operational variables necessary for experimentation. A schematic of the facility is shown in Figure 1.

![Fig. 1 - Schematic of shear test laboratory facility](image)

The data acquisition system used in the test program incorporated several pressure transducers, displacement transducers (LVDT) and a load cell.

Each “rock” sample used in the test program comprised two concrete blocks cast separately in specially fabricated 10 mm thick steel casings. The blocks remained in their casing during a test to ensure an even load distribution as well as to provide confinement. The lengths of the two concrete blocks were 250 mm and 1000 mm with cross-sectional dimensions of 280 mm x 280 mm. After curing, the ends of the two blocks were bolted.
together in such a manner that there was a 50 mm offset between the centre-lines of the two blocks. A hole was then drilled through the two blocks and a rockbolt installed as shown in Figure 2. The blocks of concrete were cast with the casings standing upright. This was to ensure the two shear surfaces would have the same level of surface roughness.

The shear test laboratory facility comprising test sample and monitoring equipment with structural modifications to the compression test machine is shown in Figure 3.

![Fig. 2 – Layout and dimensions of the test sample with rockbolt in place](image)

Fig. 2 – Layout and dimensions of the test sample with rockbolt in place

![Fig. 3 - Shear test laboratory facility showing test sample in place](image)

Fig. 3 - Shear test laboratory facility showing test sample in place

**TEST PROGRAM**

Three series of experiments were undertaken in the test program. While the Series 1 tests provided some initial results, its purpose was primarily to confirm functionality of the laboratory facility. Some modifications were made to the facility following the tests.
Six tests were undertaken in each of Series 2 and Series 3 with three strain-gauged rockbolts used in Series 3. The concrete mix was altered between the three test series in order to assess any effect of rock strength on shear behaviour. Properties of the concrete in the three test series are summarised in Table 1.

Table 1

<table>
<thead>
<tr>
<th></th>
<th>Series 1</th>
<th>Series 2</th>
<th>Series 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>65.9</td>
<td>46.9</td>
<td>68.7</td>
</tr>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>38.4</td>
<td>34.5</td>
<td>32.2</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.12</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>Cure time</td>
<td>52 days</td>
<td>31 days</td>
<td>34 days</td>
</tr>
</tbody>
</table>

A 23 mm BX rockbolt was used in the test program with typical UTS and yield strength of 335 kN and 240 kN respectively. For practical purposes, rockbolts were installed using an ARO roofbolter at Hydramatic Engineering in Newcastle using an industry-scale rig found in common use in underground coalmines. While use of the roofbolter was necessary because of the length of borehole and strength of rock, it also introduced some minor issues especially regarding repeatability of installation. The drilling arrangement can be seen in Figure 4.

![Drilling arrangement for installation of rockbolt in a test sample](image)

During the Series 2 tests, each test sample was loaded and unloaded up to four times. This process was followed as a consequence of three factors. First, the shear displacement of the laboratory facility was limited to 40 mm. Second, the strength of the rock mass/concrete was 47 MPa, the lowest in the three test series. Third, no nut and face plate was installed on the end of the most of the rockbolts used in Series 2. The combination of these factors limited the maximum shear resistance that could be developed.

The load-displacement curves for each test in Series 2 exhibited a similar trend. As shown in Figure 5, there was an initial stiff loading phase up to some transition point followed by some yielding with continued displacement.
between the shear surfaces. In one test, resistance to shearing increased with each subsequent loading cycle up to a maximum load of 350 kN whereas in other tests, the peak load reduced in subsequent cycles.

![Load-displacement graph showing four loading cycles](image)

**Fig. 5** - Load-displacement graph showing four loading cycles

![Extent of yielding in rockbolt and sustained failure of concrete around the rockbolt](image)

**Fig. 6** - Extent of yielding in rockbolt and sustained failure of concrete around the rockbolt

![Schematic indicating the amount of yielding and damage to concrete](image)

**Fig. 7** - Schematic indicating the amount of yielding and damage to concrete
In all the tests where the rockbolt was not constrained by a faceplate and nut, failure or rupture of the rockbolt was never achieved. It was thought that this was due to failure at the rockbolt/resin interface allowing slippage within the relatively short length of encapsulation. Hence the rockbolt was free to deform, limiting the amount of shear resistance and axial load developed in the rockbolt.

The relatively low strength of the concrete allowed deformation of the rockbolt to occur with failure over a significant area of the concrete around the rockbolt. The lack of fixed constraints on the rockbolt due to a combination of slippage in the rock mass and mobilisation of the pivot points meant the system was able to sustain a reasonable level of resistance to shear loading over a large range of shear displacement.

The extent of yielding that occurred and failure of concrete around a rockbolt is illustrated in Figures 6 and 7.

With a faceplate and nut attached during the Series 3 tests, a somewhat different load-displacement characteristic was observed. Figure 8 shows a typical load-displacement result for a pre-tensioned rockbolt. The top (dark grey) line indicates the variation in shear load with shear displacement. The load initially increased sharply to the pretension load indicating the system is quite stiff. Thereafter stiffness reduced as there was a gradual increase in load up to some peak value with failure occurring shortly after. The peak load sustained in the test was approximately 400 kN at a corresponding shear displacement of 45 mm. This is much higher the ultimate tensile strength and nearly double the shear strength of a rockbolt.

![Fig. 8 - Variation in shear and axial loads with shear displacement when using a pretensioned rockbolt](image_url)

The lower (light grey) line in the graph indicates the change in level of normal or axial load as measured at the load cell at the collar of the borehole some 250 mm from the shear plane. The line begins from a threshold axial load of 40 kN, equivalent to the pretension applied to the rockbolt. It would appear that there was initially little measurable change in axial loading at the collar until a displacement of approximately 6 mm corresponding to a shear load of 150 kN. Above this level, axial load increased at a rate of 6 kN/mm up to a maximum of 150 kN compared to a slightly higher rate of 7 kN/mm for shear load.

Figure 9 shows the profile of a failed rockbolt. It was found that there was appreciably less failure of the concrete surrounding the rockbolt as compared to that observed in the Series 2 tests. This is in accord with the higher strength concrete used in the Series 3 test samples. The combined constraints of stronger concrete and fixed end point of the rockbolt meant sufficient stresses could be developed to cause failure of the rockbolt.
This is in accord with previous findings that the level of induced normal force, $\sigma_n$, varies directly with applied shear force, $\tau$, such that

$$\sigma_n = \frac{\tau}{\mu}$$

where $\mu$ is the friction in the system principally the friction between the rock surfaces. Re-arranging and taking account of the clamping stress, $C_o$, we get the more usual equation

$$\tau = C_o + \mu \sigma_n$$

A set of tests with three strain-gauged bolts was undertaken to examine the variation in load with distance along the rockbolt from the shear plane. Unfortunately, due to a combination of issues associated with installation, orientation of the bolts, gauge alignment and instrumentation little quantifiable results were obtained. The strain gauge readings during a test fluctuated widely with strains in excess of ±15,000,000 microstrain being measured. During earlier calibrations tests, approximately 500 microstrain was measured with an applied axial load of 100 kN. Further work will be needed to determine the cause for this discrepancy.

Although the magnitude of values are questionable, Figure 10 shows there was some correlation especially between the level of strain along the rockbolt and the level of applied shear load and shear displacement.
FAILURE OF THE REINFORCING ELEMENT

An examination of all failed rockbolts indicated they did so in a ductile manner with necking evident as can be seen in Figure 11. Ductile failure occurred between the two plastic hinges (bending regions) on either side of the shear plane associated with deformation caused by the shear displacement; this is where the bending moment was greatest. Between the two hinge points, the loading regime was altered such that given sufficient shear loading, the rockbolt failed axially in tension. This would account for the higher failure loads observed that were well in excess of the shear strength of the rockbolt.

![Fig. 11 - Profile of a rockbolt that failed due in shear](image)

Inspection of the failed surface of the reinforcing element confirmed the failure mechanism as being a typical ductile bending, necking and then tensile failure. The failure initiated in the centre of the necked region with the crack, then progressed laterally towards the edge of the element in the area known as the radial zone. The fracture is then completed via a shear lip on the outer extremities of the element. A reinforcing element that is subject to a pure axial load creates a symmetrical shear lip around the outer edge of the failed rock bolt section as indicated in Figure 12, whereas the shear lip in the failed element subject to a shear load creates a more ellipsoidal shape, engaging at the upper and lower section of the element. The shear lip is negligible at the sides of the element where the applied shear load is perpendicular to the element.

![Fig. 12 – View of the failed cup surface](image)

The development of this unique shear lip can be due to the final rupture of the rock bolt occurring at the ends where maximum stress is located in this section of the reinforcing element. When a shear load is applied to the element, the greatest stress within the rock bolt is located in the same plane as the applied load where the element is subjected to a tensile and/or compressive stress at either extremity. This final rupture of the element due to the
shear lip occurs predominately in the same plane where the shear load is applied, compared to the uniform smooth annular area formed adjacent to the free-surface of the element when subjected to a pure axial load.

To further analyse the failure mechanisms within the reinforcing element, scanning electron microscope (SEM) analysis was undertaken of the fracture surface of the failed element by the School of Materials Science and Engineering (UNSW). The two SEM results indicated the phenomenon of a dimpled rupture, which occurs via the process of microvoid coalescence. The two fractures started in the centre of the section of the reinforcing element and then radiated outward. Once the crack was near the surface the stress state changed from triaxial to plane strain and this was responsible for the change from flat face fracture that is perpendicular to the tensile axis, to slant fracture (45 degrees to the tensile axis) that produces the shear lip (Crosky, 2005).

CONCLUSION

In summary, the main findings of the test program were as follows.

1. A standard BX rockbolt exhibited a greater resistance to shear loading than had been anticipated; greater than both the ultimate tensile strength (UTS) and shear strength of the individual rockbolt element. The amount of shear displacement and deformation of the rockbolt was much greater than had been expected; nearly double that which had been allowed for in the initial design of the laboratory facility.

   Failure loads of up to 400 kN were observed compared to typical UTS values of 250-300 kN.

   This result emphasises that behaviour of the complete system is not solely a function of the individual elements that make up the system such as the reinforcement elements. Rather behaviour is significantly influenced by the interaction that occurs between the system’s various components such as the rock reinforcement elements and the rockmass.

   One potential ramification of this finding is that the extent of resistance to shear and the degree of deformation which is allowed for in the design of underground support systems may be well underestimated.

2. Strength of the rockmass was shown to affect the performance of the system. In tests using higher strength concrete, the amount of shear displacement was less than that observed in comparable tests with weaker strength concrete samples. Stiffness of the system increased with strength of the rockmass. Conversely, maximum load resistance decreased with rockmass strength.

   The stronger concrete is thought to have limited the extent of the “activation zone” along the rockbolt. Less crushing of the concrete about the rockbolt was observed in the stronger concrete samples indicating the material was less compliant.

   Hence in design of an underground support system, cognisance must be given to the strength parameters not only of the rockbolt but also of the rockmass. The result indicates that in endeavouring to design for a level of performance account has to be made of the rockmass, for example:

   • in strong rock, the support system is likely to be less compliant and stiff. The system is better able to maintain integrity of the laminated beds and hence contribute to overall stability.
   • in weak rock, the system is likely to be more compliant and allow for more differential movement between bedding plains. Conversely the strength of the system would be enhanced as the rockbolt is capable of sustaining a higher resistance to shear load than can be achieved in a stronger rockmass.

3. The performance of a rockbolt subjected to shear loading as characterised in a plot of applied shear load versus shear displacement demonstrated two distinct zones of behaviour. Initially, the system was relatively stiff with resistance increasing dramatically with very little shear displacement up to some level of load beyond which yielding was observed until the rockbolt eventually failed.

4. Stiffness of a system is unaffected by cyclic loading. In earlier tests, with the limited shear displacement capacity, load on the sample had to be temporarily withdrawn to allow packers to be installed and the load was re-applied. The load-displacement curve was found to follow a similar path as in continuously loaded tests.
Hence cyclic loading and unloading to less than the yield point is unlikely to impact performance of the rockbolt support system.

5. Over the range of loading rates examined, stiffness of the system varies with the rate of load application; higher loading rates result in greater stiffness.

6. Examination of the fracture surfaces of the failed rockbolts showed the rockbolts failed in a typical ductile manner and not in a manner usually associated with shear failure. Failure was initiated in the centre of the necked region of the element with cracks radiating outwards towards the surface. The fracture was completed via a shear lip on the outer extremities of the element.

7. Although a rockbolt may have failed axially at the rockbolt/resins interface, it may still be capable of offering appreciable resistance to shear loading and hence provide some support to the rockmass.

In tests where no face plate and nut were used at the collar of the borehole, the rockbolt generally could not be made to fail. At some point during a test, the limited length of encapsulation in the shorter block was insufficient to react against the axial load generated in the rockbolt resulting in failure of the rockbolt/resin interface. With continued shear displacement between the two test blocks, the level of resistance remained constant as the rockbolt was extruded through the borehole as it was not fixed or constrained by any face plate.

8. Pre-tensioning of a rockbolt increased its initial resistance to shear displacement. When pre-tensioned, a rockbolt initially exhibited a high level of stiffness. With continual loading, a point was reached when shear displacement increased with load at a rate similar to that observed in untensioned elements.

The magnitude of load necessary to initiate shear displacement increased with the level of pre-tension.

Hence pre-tensioning is beneficial to increasing the stiffness of a rock support system dependent on the level of pre-tensioning.

9. Use of strain-gauge rockbolts confirmed that shear loading generated an axial tensile load in the rockbolt, effectively clamping the shear surfaces together. The level of axial or normal load increases with shear load. This resultant normal force activates the frictional forces between the two rock surfaces that enhance resistance to shear loading.

When the orientation of the strain gauges was aligned with the shear plane, failure of the rockbolt was initiated in the corner of the one of the longitudinal slots of the strain-gauged rockbolt where the bending stress is at a maximum. Here plastic hinges are created that fractured the rockbolt.

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REFERENCES

AN ASSESSMENT OF LOAD TRANSFER MECHANISM USING THE INSTRUMENTED BOLTS

Hossein Jalalifar¹, Naj Aziz², Muhammad Hadi²

ABSTRACT: Load transfer capacity and failure mechanism of a fully grouted bolt installed across joint plane in shear is evaluated experimentally and numerically, tests were made on un-instrumented and strained gauge instrumented bolts. Four types of bolts, with different properties and surface configurations were selected for the study. The changes in strength of the concrete, bolt mechanical properties and bolt pretension load were evaluated in different shear environments. Results from the instrumented bolts and numerical simulation showed that the tensile and compression stresses and strains are generated at early stage of loading and hinge points are created at both sides of the shear joints. Strains are in greater value in vicinity of the shear joint. The failure location moves towards the bolt joint intersection due to the increasing shear load, shear displacement and axial load developed along the bolt.

INTRODUCTION

Rocks bolting is the most common form of ground support in use in both civil and mining excavation engineering and are used for ground reinforcement as both temporary and permanent systems to support the ground. The efficiency of the reinforcement system depends on the load transfer mechanism and the shear stress sustained at the bolt - joint interface. Factors influencing the shear resistance are; bolt diameter, hole diameter, steel quality, confining pressure, and concrete strength (Bjurstrom 1974), (Azuar 1977), (Hibino and Motojima 1981), (Dight 1982), (Spang and Egger 1990), (Ferrero, 1995), (Grasseli 2004).

In the previous extensive series of experimental tests, in double shear method four different concrete strengths, 20, 40, 50, and 100 MPa and various pretension loads 0, 5, 10, 20, 50 and 80 kN were used. From the analyses it was found that:

- Bolt profiles plays a significant role in load transfer mechanism,
- Bolt pre-tension increases the level of shear resistance,
- The resistance of the bolt will depend on the strength of the concrete
- Increasing the strength of the concrete reduced joint shear displacement and increased shear stiffness.

To gain a clear understanding of the pattern of load and stress build up along the bolt, two tests from the extensive number of above tests were carried out on strain gauged instrumented bolts (both of Bolt Type T2), one test was made with a bolt not subjected to pretension load (zero pretension) and the other with a pretension load of 20 kN. During the shearing process, the bolt was deformed with joint displacement. The longitudinal axis of the sheared bolt is deformed into a curve producing a lateral shear load, an axial load, and two critical points: one in bolt-joint intersection and the other at the hinge point. These loads produce stress resultants in the form of bending moment, shear and axial forces throughout the beam at the bolt - joint intersection or at the hinge points. In addition, several tests were carried out until bolts reached to failure and necking and cutting were appeared.

EXPERIMENTAL STUDY

Laboratory tests were carried out in four types of bolt. These were bolt Types T₁, T₂, T₃ and T₄ (high strength steel and low strength steel). The tests were carried out in three-piece pre-cast concrete blocks, of strengths 20, 40, and 100 MPa respectively. Concrete was used to simulate different rocks, as it was easier to prepare and to simulate different strengths. Minova PB1 Mix and Pour resin was used to install bolts in precast and reamed holes in different strength concrete blocks. Bolt and hole diameters were proportioned to maintain resin thickness encapsulation constant. Figure 1 shows the profiles of various bolts used in the study. Bolt characteristics are shown in Table 1.

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To achieve a meaningful result, various tests were carried out including:

- Shear testing of the bolts in high capacity compression machine (5000 kN), thus allowing the bolts to snap at the sheared joint plane.
- Shear testing of the instrumented bolts for determining the hinge points position and strain built up along the bolt.
- Testing of the bolts in higher strength concrete.
- Shear testing of lower strength steel bolts, which allowed the bolt to fail at much lower shear loads.

Figure 2 shows the general set-up of the assembled double shear box in a 500 tonne capacity compression testing machine and the photographs of different deformed bolts. Tests were made with and without pretension loads of 5, 10, 20, 50 and 80 kN.

### Table 1 - Bolts characteristics

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Rib Spacing (mm)</th>
<th>Core diameter (mm)</th>
<th>Rib height (mm)</th>
<th>Max. Tensile load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T₁</td>
<td>11.5</td>
<td>21.7</td>
<td>1.0</td>
<td>328</td>
</tr>
<tr>
<td>T₂</td>
<td>11.5</td>
<td>21.7</td>
<td>1.5</td>
<td>342</td>
</tr>
<tr>
<td>T₃</td>
<td>1.4</td>
<td>10.3</td>
<td>0.6</td>
<td>44</td>
</tr>
<tr>
<td>T₄</td>
<td>7.74</td>
<td>11.7</td>
<td>0.8</td>
<td>67</td>
</tr>
</tbody>
</table>

**Fig. 1 - Bolts profile configuration**

**Fig. 2 - (a) General set-up of the assembled double shear box in a high capacity testing machine (5000 kN)**

**Fig. 2 - (b) Post test deformed bolts and resin encapsulation**

**DOUBLE SHEARING OF INSTRUMENTED BOLT**

Figure 3 shows the location of the strain gauges in Bolt Type T₂. In each location, designated 1 to 6, strain gauges were mounted on opposite sides of the bolt and single strain gauges were installed at locations 7 and 8, situated beneath the bolt. The spot where each strain gauge was located had the bolt profile ground flat and smooth. The
21.7 mm core diameter bolts were installed in 27 mm holes as per previous tests. Both tests were carried out in 40 MPa concrete. The strain gauge measurements revealed that both the tensile and compression stresses were generated longitudinally during shearing.

By comparing the axial strain at each location along the bolt, axial stress could be determined by the following equation:

$$\sigma_{aij} = E_b (e_{ai} - e_{aj})$$

and the shear stress distribution can be given by:

$$\tau_g = \frac{\sigma_{aij} A_b}{2\pi l} = \frac{E_b (e_{ai} - e_{aj}) r}{2l}$$

Where:
- $\sigma_{aij}$ = Change in axial stress between two adjacent gauges
- $E_b$ = Bolt modulus of elasticity (MPa)
- $e_{ai}$ = Axial strain at gauge 1 (με)
- $e_{aj}$ = Axial strain at gauge 2 (με)
- $l$ = Distance between gauges (mm)
- $r$ = Bolt radius (mm)

![Fig. 3 - Schematic diagram of the strain gauges locations in the reinforcing element (a) without pretension load and (b) 20 kN pretension load](image-url)
Using the above equations in zero pretension conditions, it was found that, for a 30 kN shear load, the maximum tensile and shear stresses, between the strain gauges 3 and 4 at the bolt / grout interface were 196 MPa and 35 MPa respectively. Beyond this load, the stresses were reduced, indicating the bond failure between bolt and grout. The minimum axial and shear stresses were recorded at 50 kN shear load, which are approximately 18 and 3.25 MPa respectively. This situation occurred at the elastic region of the shear load-shear displacement curve, which was supported by both experimental and numerical results. Figure 4 shows the variation of the strain changes along the bolt.

The following were observed:

- Strain gauge No 3, located in the compression zone and placed 60 mm away from the shear joint, produced 2.5 % strain at 60 kN shear load (one half of the total shear load acting of two joint planes) at zero pretension load. This value of strain is in the range of the plastic region (higher than 0.3 % at the end of the elastic region). The yield situation occurred around 20 % of the maximum tensile strength of the bolt.
- The formation of two plastic hinge points in the bolt located symmetrically opposite either side of the sheared joint plane was determined by the strain measurements. Beyond the hinge point and towards the end of the bolts there was a gradual decline in the rate of strain. This was in line with the findings obtained from the numerical simulation. For the strain gauges located near the hinge points it was found that very small shear load (12 kN at strain gauge no 5) was required to strain the outer profiles of the bolt. Thus it was clearly evident from Figure 4 that both the tensile and the compression zones were initiated in the bolt during the early process of shearing.
- For the pretension case, it was found that the hinge point was located around 30 mm from the shear joint. The location of the hinge points was dependant upon the strength of the concrete. In weak concrete, there will be excessive crushing of the concrete in the vicinity of the sheared joint faces leading to greater distance between the hinge point and joint spacing. However, the hinge point location will be closer in high strength concrete. Figure 5 shows the strains developed along the bolt in 20 kN pretension load. Thus it is reasonable to assume that the location of the hinge points are likely to be in these zones and this finding is in agreement with the numerical studies.

![Figure 4](image-url)  
**Fig. 4 - Strain rate along the bolt, as measured on the bolt, in zero pretension load**

![Figure 5](image-url)  
**Fig. 5 - The variation of the strain gauge measurements along the bolt at 20 kN pretension load**
SHEAR LOAD BUILT UP AT BOLT FAILURE STAGE

Next, a series of tests were carried out to examine increased shear displacement until the bolt completely sheared (failed). Two approaches were adopted:

- Shearing a small diameter bolt. The bolts were Bolt Types T3 and T4, tested in 40 MPa concrete.
- Shearing a 23 mm diameter bolt in 100 MPa high concrete. Only Bolt Type T1 was tested.

The above tests were undertaken at different confining pressures similar to tests carried out under limited displacement. During the shearing process the shear displacement is increased, lateral and axial loads are developed along the bolt and surrounding materials. The factor of shear resistance may be the resultant of both lateral and axial loads due to the bolt deflection.

Hinge points were created in the bolt at both sides of the shear joint plane, with the gap being increased between bolt-grout and grout-concrete. Grout was completely damaged at compression zones and concrete was fractured along the bolt axis in all three blocks. The failure process of the system appears to be influenced by concrete strength. Bolt failure in strong concrete occurred in shear at the shear joint plane between the hinge points. In weak concrete, no bolt failure occurred as the bolt cuts through the concrete.

Figures 6 shows the load displacement profiles of the bolts tested under different axial load conditions in bolt Types T3 and T4. The level of maximum shear loads and displacement were different because of different pretension loads. Failed sheared Bolt Types T3 and T4 are shown in Figures 7 and 8 respectively.

Figure 9 shows the load displacement profiles of the bolt Type T1 in different pretensions in 100 MPa concrete. Figure 10 shows the failed bolt across the joint planes in Bolt Type T1 in 100 MPa concrete. Figure 11 shows the longitudinal view of concrete blocks after failure in 100 MPa concrete. The bolt failure is located in the vicinity of the bolt-joint intersection, between the hinge points. Moreover, the bolt imprint on bent resin grout shows the grout being overwhelmed due to the compression stresses. The reaction forces are distributed about 60 mm in the bolt and away from the shear joint plane in the outer blocks. In this zone the concrete is extensively fractured.

Fig. 6 - Shear load versus shear displacement in 0, 5 and 10 kN pretension load in Bolt Types T3 and T4 in 40 MPa concrete
Fig. 7 - Bolt failure view in different pretensioning

Fig. 8 - Bolt failure angle surrounded in concrete 40 MPa and 18 mm hole diameter in Bolt Type T

Fig. 9 - Shear load versus shear displacement in 100 MPa concrete and various pretensions in Bolt Type T₁
Fig. 10 - Failure zone in bolt type T₁ in concrete 100 MPa and 80 kN pretension load

Fig. 11 - Failure location in Bolt Type T₁ surrounded in concrete 100 MPa and 27 mm hole diameter in full details of failure process
**SHEAR LIP GENERATION**

Figure 12 shows the side profile of the failed rock bolt embedded in 36 mm hole diameter and 20 MPa surrounding concrete in 20 kN pretension load. Inspection of the failed bolt showed that the failed surface was caused by the axial and shearing failures, which appears to be initiated with small cracks originating from the center. However, it is suggested that after the yield point, the shear stress generation in the vicinity of the shear joint, through the reinforced bar is almost constant and the bolt fails with the increase of the axial load along the bolt due to excessive bending with combination of shear load developed. It shows the shear lip in the failed bolt has created an ellipsoid shape.

The ratio of axial load developed along the bolt over ultimate tensile strength of the bolt versus shear displacement in concrete 100 MPa with 80 kN pretension load is shown in Figure 13.

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**EXPERIMENTAL OBSERVATIONS FROM FAILED BOLTS**

- The snapping or failure of a bolt across joint planes was the result of shearing and tensile loading because the shear surface was not vertical and parallel to the vertical joint planes. The surface angle was 12° from the joint plane in bolt Type T3 as shown in Figure 8.
The peak elastic yield point “P” has gradually moved closer towards the bolt-joint intersection.

For a pre-tension load of 80 kN, shear displacement at failure for Bolt Type T4 was 40% higher than the corresponding shear displacement for Bolt Type T3 (see Figure 6).

The displacement rate of the sheared block in 100 MPa strength concrete was lower than in both 20 and 40 MPa concrete, as expected.

The failure load for Bolt Type T1 with a pre-tension load of 80 kN was 799 kN. This was in excess of the axial tensile failure load of the bolt at 340 kN.

Crushed zones in 100 MPa concrete were less than in 40 MPa concrete. The length of the crushed zone was 60 mm either side of the joint plane which demonstrated significant resistance from the concrete and less vertical displacement during shearing.

During shearing the bolt failed at around 66% of its maximum tensile strength. This failure could not have occurred purely due to axial loading, which again demonstrates that failure was a combination of shear and axial loads at the bolt-joint intersection (see Figures 13).

3D FEM SIMULATION

Using ANSYS, version 9 (ANSYS, 2003), three-dimensional simulation of the bolt shearing process was carried out to examine the behaviour of bolted rock joints in relation to the experimental results. Simulation of several models in varying conditions (a range of bolt tensile load and concrete strength) was carried out under a progressive vertical load and results were analyzed for both linear and nonlinear regions of the load-deflection behaviour. From the analyses it was found that, the stresses in the upper convex section of the bolt are in tension, while the lower opposite side are in compression. This situation will occur in reverse on the concave section of the bolt in the other side of the shear joint plane. In addition, the numerical simulation showed that the tensile stress in the bolt was increased and expanded towards the shear joint with increasing the pretension load and bolt deformation.

The distribution of the axial strain and shear stress along the bolt in the vicinity of the sheared joint is drawn in Figure 14a and b respectively. It shows that higher strain is generated from the hinge points and propagates towards the bolt-joint intersection. The maximum shear stress is concentrated at the bolt joint plane intersection. Pretension caused a reduction in bolt shear stress. With increasing shear load and bolt deformation, axial stresses are expanded and moved towards the shear joint location. The combination of shear and axial stresses, the bolt will fail at the joint plane area, as it was discussed in the experimental section. At the post-elastic yield point, the shear stress was almost constant and unaffected by the increase in both the shear and pretension loads. This behaviour occurred at approximately 4 mm of bolt deflection in 20 MPa concrete. The shear stress diagram for all concrete strength, had the same trend, but the value of shear stress was found to decrease with increasing the pretension load.

Softer concrete has experienced higher strain along the bolt and the value of induced tensile strain was higher than the compressive strain. Induced strain is increased with increasing the shear load. With increasing pretension load in post failure behaviour there was no significant changes in strains along the bolt. However, the area of tensile strain has expanded and compression strain reduced. The high level of stress produced in the concrete caused fractures and failures in the vicinity of shear joint region. The zone of high stress concentration in weaker concrete is significantly broader, at 90 mm from the shear joint plane, in comparison to 60 mm length, obtained for 40 MPa concrete.

When the shear load increases, grout will break at the tensile zone and overwhelmed at compression zones. This situation usually will start at the bolt elastic behaviour region and progress beyond the yield point, and the grout will separate from the bolt at tension zone in the vicinity of the shear joint. Due to the axial bolt load, yield in the grout can be determined when the actual bond stress, between bolt and grout, is equal to the grout yield strength. The plastic strain is generated along the grout layer, in particular between the hinge points, when the induced stresses exceeds the grout strength. The values of strain in the grout layer, in plastic state, is ten times greater than at the linear region, particularly at the critical zones in the vicinity of shear joint. Obviously the grout under such severe condition will be fragmented.
CONCLUSION

The double shear testing represents a useful method of assessing the bolt behaviour in a stratified reinforcement. The evaluation of the shear strength of rock joints reinforced with fully grouted bolt was analysed both experimentally and numerically. The main conclusions drawn were as follows.

- Bolt will fail in vicinity of shear joint plane, between hinge points, due to the combination of both the shear and the axial loads.
- Tensile and compressive stress zones along the bolt are located on each side of the shear plane. The nature of stress concentration is dependent upon the deflection direction of the sliding blocks relative to each other.
- Higher value of shear stress contours occurs along the shear plane.
- The amount of bolt shear displacement due to differing pretension loads is insignificant at the elastic range. However, this is more significant after the yield point, as demonstrated by both experimentally and numerically.
- In all type of bolts tested experimentally, the shear load of the bolt, in general, increased with increasing bolt tension.
- The distance between hinge points was reduced with increasing the strength of material. However, there are no significant changes in hinge point distance with increasing pretension effect.

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REFERENCES

AIRBORNE AND TERRESTRIAL LASER SCANNING:
APPLICATIONS FOR ILLAWARRA COAL

Peter Riley¹ and Peter Crowe¹

ABSTRACT: Laser Scanning technology was introduced to the mapping industry in 1998. The technology enables the acquisition of point data by a laser scanning device mounted either in a fixed or rotary winged aircraft (Airborne, ALS) or placed on ground stations (Terrestrial, TLS). Laser scanning provides a significant increase in data points (e.g. 18 million data points can be gathered per hour) over traditional data capture methods. It also affords cost reductions due to acquisition and processing times when compared to aerial photogrammetry or ground surveying, particularly in areas that are densely vegetated or difficult to access. Due to the relatively quick acquisition and processing time, ability to filter vegetation from the datasets and a simple, flexible, data format, ALS has become an integral extension to the mapping tools available to Illawarra Coal.

INTRODUCTION

Airborne Laser Scanning (ALS or LIDAR) was first demonstrated in 1993 and was introduced to the mapping industry in Australia in 1998. The technology is a further development of the Airborne Laser Profilers used by the forestry industry for many years (Hansen and Jonas 2001, Wever and Lindenberger). The technology involves the acquisition of three dimensional ground point data by utilising a laser scanning device operating at frequencies in the near infra-red. The scan direction is orthogonal to the flight path. The unit is mounted in a fixed or rotary winged aircraft flying at altitudes of up to 1000 m. Spatial position is controlled by kinematic dual frequency GPS and the airborne platform's attitude is monitored by the use of an inertial navigation or measuring system (INS/IMU).

The technology can produce data coverage over a large area very rapidly and in terrain that would preclude aerial photography or ground surveys. The ALS acquisition can also occur at night or under cloud in low light if required. The need for a spread of three dimensionally surveyed ground control points in the form of aerial targets is also unnecessary. It is necessary however to provide some ground truth correlation but this can be achieved post survey and in areas of the survey that are easier to access.

The mining industry has traditionally used ALS for use as an exploration broad acre terrain modelling tool. It has also been utilised in new mine site planning and design. In the open cut environment it has been used for slope and stability assessment. The ease of acquisition and rapid processing has made it useful in overburden stripping and volume determination. It has proved to be useful in repeat surveys for environmental monitoring and more recently it has been applied in subsidence monitoring using isopach analysis to detect surface changes.

Illawarra Coal has utilised ALS data to produce GIS layers that, in combination with other data (e.g. Aeromagnetic surveys), can enhance the interpretation and presentation of selected surface features that may be of geological significance to mine planning.

Data sets derived from ALS produce Digital Elevation Models (DEMs). These are used in analytical operations such as slope and aspect determination and surface modelling. These data sets can be filtered to create Digital Terrain models (DTMs), Triangulated Irregular Networks (TINs) and contour lines (at varying intervals depending on requirements). Regular grid products such as raster coverages and datasets that facilitate the monitoring of change management in the man-made and natural environments can be created.

ALS does not produce imagery but most operators will provide a digital image (from conventional cameras) if requested. It is possible however to produce a true-to-scale image of the surface by applying a greyscale reading to the "intensity" of the return signal. Figure 1 shows that like-features such as roads can be extracted from the dataset based on the intensity value.

¹ BHP Billiton, Illawarra Coal
The laser beam operates by sweeping the terrain in a swathe to deliver a point cloud. Depending on the requirement, point data can be collected at spacings from sub-metre to 10 metres. Swathe widths are generally up to 700 metres.

When acquiring data in densely vegetated areas, the laser (operating at 1.04 micron wavelength) will receive multiple returns from single pulse travel which will capture tree tops as well as ground. Algorithms have been written to determine which points represent the ground or non-ground. This technique can allow discrimination of vegetation or specific features such as transmission line conductors and tree height. Illawarra Coal have utilised this data in the selection of exploration drill sites that require rigs to be airlifted.

The manufacturers stated accuracy of the ALS positioning is in the order of 10 - 15 cm (rms) in the vertical and 50 - 100 cm (rms) in the horizontal. Illawarra Coal has conducted ground survey checks of specific points and have identified that the standard deviation of the results is within these limits. To obtain this accuracy however a GPS reference base station must be within 50 km of the operating aircraft.

**TERRAIN MODELLING**

ALS technology essentially provides only one product. That is, a point cloud describing the earth surface (Maas, 2001). To successfully describe the earth’s surface each point must have three dimension values, that is, X, Y & Z. The ALS data also has an additional observable, the intensity value of the reflected signal. This can be used to extract features based on the intensity of the return beam. Data produced from ALS surveys are in digital format delivering Easting, Northing, and Height values of each laser target. This simple data format allows for ease of data flow into software to produce a wide range of models and data products. Typical software includes Geographic Information Systems (GIS), Computer Aided Design (CAD) and other industry specific modelling software such as Minescape.

The software typically takes the point data and employs an algorithm to create a model of the earth’s surface.

These models can be continuous surfaces such as a TIN, shown in Figure 2 or Raster, Figure 3, or a Digital Elevation Model (DEM) such as a regular point grid or contour lines. The operator can specify cell size or grid spacing and interpolation method. These products then form the basis for further investigation and analysis of the environment in which mining will take place.

The resultant products can then be used to visualise the surface of the earth, calculate earth movements from repeat surveys, calculate drainage patterns for watersheds or create linear profiles. Figure 4 below shows a surface profile between two points to assess line of site.

The scanning beam from the ALS transceiver cannot initially distinguish between ground points and other points above the ground such as trees, buildings or power lines. Sophisticated algorithms must be applied to the data to separate ground from other objects (non-ground). The non-ground points can be used to identify and visualise the objects on the surface, such as the tree canopy, buildings or powerlines (Figure 5).

The ability to quickly filter ground from non-ground points is one of the many advantages of ALS over traditional aerial photography. For the mining industry this is beneficial because the shape of the surface can be benchmarked before any mining has taken place.
Fig. 2 - Triangular irregular network (TIN) of the Mt Kembla area, produced in ArcGIS by BHP Billiton Illawarra Coal

Fig. 3 - A raster surface over approximately the same area as above. Used more often for surface calculations and the creation of other products than for visualisation. Produced in ArcGIS by BHP Billiton Illawarra Coal
GEOLOGICAL STRUCTURE INVESTIGATION

Using a technique called Residuals Mapping, geologists can identify surface features that may otherwise go undetected when using aerial photography or small scale contour maps, or even field visits. The process involves taking the ALS point dataset and applying smoothing interpolators to the surface. This eliminates irregular changes in grade.

The smoothed surface is then subtracted from the original dataset (Figure 6) leaving only those parts of the surface that varies by a small amount (residuals) from the general topography. Sometimes these residuals form patterns (Figure 7) that may indicate underlying geological structure.

Illawarra Coal has used this technique to model surface lineaments associated with dykes. This method creates efficiencies in the planning and execution of exploration work by accurately targeting likely locations before the team enters the field.
Many groups and authorities in the community are vitally interested in the effect that mining activities have on infrastructure and the natural environment. Public authorities such as the Department of Primary Industry (DPI) and the Dam Safety Committee (DSC) monitor and regulate the mining activities in the Southern Coalfield, while community groups watch carefully the effect on their local environment. The issues are often emotive and much effort is invested by all stakeholders to ensure that sufficient, pertinent and reliable data is collected.

In the Illawarra and Macarthur regions, a major issue to stakeholders is ground subsidence as a result of Longwall Mining. Subsidence can potentially affect infrastructure such as gas and water pipelines, roads and houses, as well as the natural environment, particularly river beds and cliff lines.

Data is regularly captured in sensitive areas in order to monitor and report to the major stakeholders. Traditionally this data has been collected by field survey or to a lesser extent by aerial photography. The field survey data capture is cheaper than aerial photography but is limited spatially due to the time to acquire the data and surface access restrictions. Photogrammetric methods are costly and still need tight ground control to be accurate enough to achieve the desired tolerances. Both of these methods are also restricted by weather and the environment (e.g. the rugged terrain and thick rain-forest of the Illawarra Escarpment).

An ALS survey is ideally suited to the data capture requirements in these rugged and densely vegetated environments. By utilising ALS to monitor subsidence movement Illawarra Coal can provide a data set to their stakeholders within a reduced time-frame and on a much larger scale than could ever be achieved by conventional surveying techniques.
A recent study conducted by Illawarra Coal, in conjunction with the University of Wollongong, demonstrated the potential benefits of ALS data for subsidence mapping (Palamara et al 2005).

In the study, the results of two pre-mining ALS surveys were compared to a post-mining survey to map the extent and magnitude of subsidence in the rugged and heavily wooded landscape. Figure 8 above shows the terrain immediately west of the Illawarra escarpment.

The results of the study, Figure 9 below, showed that the mean vertical error was well within acceptable limits for assessing regional subsidence movements due to coal mining operations. The figure below shows the extent and magnitude of the subsidence only. It does not show the true shape of the topography.

The study concluded that the assessment of watercourses, steep slopes and cliff areas should be treated with caution during height change analyses using ALS because the horizontal accuracy of ALS is generally not suitable for the accurate mapping of steep or narrow features (French 2003 in Palamara et al 2005).

CONTOUR MAPPING

The creation of contours over areas of more than a hectare or two has traditionally only been available using photogrammetric methods. Providing contour maps to the mining industry, surveying companies and government agencies is an expensive exercise and contours with intervals of less than 10 metres have long been on the wish lists of most mining companies.

The ability to create engineering contours ‘on demand’ for areas of the customer’s choosing enables rapid response to the requirements of the exploration program or changing business needs. ALS point data, in conjunction with appropriate software, allows such flexibility.
Fig. 9 - Subsidence model showing exaggerated magnitude and location of subsidence. The model was produced in Surfer 8 using a subtraction algorithm.

Fig. 10 - One metre contours over part of Appin NSW. Created in ArcGIS 3D Analyst

Caution should be exercised in the creation of contours. While the software makes it possible to create contours with an interval as little as 0.1 m, the data may not be designed to perform to that accuracy. It is essential that the survey be designed to meet the requirements of the task.

High resolution surface contours, in the order of 1 metre, over proposed mining areas, Figure 10 above, are becoming the desired mapping requirement for designing future exploration activities. Geologists are able to look at the surface, stripped bare of all vegetation, and identify surface features such as outcrops or depressions that may indicate an underlying geological structure. During field trips these features can then be found with a high degree of confidence using coordinated positioning by means of a hand-held GPS.
High resolution surface contours contribute significantly to the creation of accurate predictive borehole logs. A predictive borehole log is required before a borehole is drilled and generally modelled on data from adjacent existing boreholes. These logs provide a variety of information for drillers, geologists and planners about the substrata horizons or conditions that may be encountered during drilling.

**TERRESTRIAL LASER SCANNING**

Terrestrial Laser Scanning (TLS) uses the same principles as ALS, except that it is ground based. Locating the scanner on the ground gives some distinct advantages for capturing discrete objects from multiple angles.

These systems can measure several thousand points per second allowing for data sets to be collected far in excess of that which could be obtained by traditional surveying or photogrammetric techniques.

TLS is most useful for capturing small (relative to those captured from an aircraft) irregular objects such as buildings, earthworks and landforms such as cliff faces which can be profiled and monitored during mining.

Illawarra Coal has successfully used TLS for monitoring and documenting natural structures and sites influenced by long wall mining.

The TLS surveys can gather information in a non-invasive manner from a safe location. Traditional surveys require people to disturb sensitive areas or subject themselves to dangerous work environments to obtain the required data. The data sets are delivered with high accuracy.

The laser is classified as a Class 2, green laser which is safe to operate indoors and outdoors and therefore does not impede ongoing work in the vicinity. The instrument can be operated in any lighting conditions.

Other applications this technology is useful for include:

- Surface modelling and volume calculations
- Cavity measurement (e.g. Aboriginal rock art sites)
- Slope stability studies
- As-built surveys (including detailed measurements for fabricating plant and machinery that may not have engineering drawings)
- 3D visualisation of structures

An important recent project was the monitoring of a site on the Georges River near Appin in NSW. This site is significant to the local community and it was important to monitor and record changes due to undermining. The TLS scanner was deployed on 20 stations around the site, Figure 11, and data was captured and integrated into a model to be used for analysis.

Analysis algorithms are the same as those used with ALS data. That is, cleaning the data by removing any extraneous points, and then comparing multiple epochs to measure changes.

After cleansing and integration, as shown in Figure 12, the data can be displayed and analysed with various software packages. Typical outputs include cross-sections of the structure, in this case overhangs and channels, contour maps and residuals maps showing where change has occurred. The finished model was also overlaid with digital photography to provide presentation material for community consultation.

**AERIAL IMAGERY**

ALS technology does not produce conventional imagery as one of its outputs. It can however, be operated in conjunction with other sensors such as large format analogue and small format digital cameras, thermal and multispectral sensors, satellite and video imagery to produce complimentary datasets (Earls and Jonas 2004). The combination of these various products with ALS data, Figure 13 below, results in a very powerful product. While it is not essential that data from different sensors is captured simultaneously, the addition of a standard or digital camera during the ALS survey can also offer significant cost savings. Unfortunately this is not always possible due to the often different acquisition parameters of the two technologies (Earls and Jonas 2004).
Fig. 11 - Un-integrated raw TLS data, displayed using intensity values to give an image-like result

Fig. 12 - The completed model of the site
Mining companies now operate in an era of increased regulation and public scrutiny. It is essential that they investigate innovative technologies that offer fast, reliable and comprehensive data capture methodologies. Airborne and Terrestrial Laser Scanners meet these requirements, providing flexible, rapid, cost effective and high quality data sets that enhance the mapping and reporting capabilities of the company.

Illawarra Coal has found that ALS and TLS are powerful tools with many proven and potential applications. They compliment, rather than replace a wide range of existing data capture methods and have proven that they add value to the interpretation and presentation of existing data. Ongoing developments such as the recording of the full waveform of the signal can only increase the potential of this technology.

REFERENCES


HIGH-RESOLUTION TOPOGRAPHIC DATA FOR SUBSIDENCE IMPACT ASSESSMENT AND SMP PREPARATION: METHODS AND CONSIDERATIONS

Daniel Palamara¹, Gary Brassington², Phil Flentje¹, Ernest Baafi¹

ABSTRACT: Corporate and social requirements relating to sustainable mining practices have resulted in an increasing need for identification and assessment of natural features that may be susceptible to coalmine-induced subsidence. Natural features such as, cliff lines, watercourses and steep slopes, that are typically susceptible to subsidence-induced impacts can often be identified and quantified using high-resolution topographic data and a geographic information system (GIS). Once identified, digital representations of these features can be used in the impact assessment process and for Subsidence Management Plan (SMP) preparation.

This paper demonstrates the use of topographic data for site characterisation and feature identification purposes by mapping susceptible areas for a study site, including valley floors, steep slopes, drainage lines, and erosion-prone areas. It also discusses the potential use of topographic data and GIS for assessing subsidence impacts through knowledge- and data-driven approaches. The assessment of pre- and post-subsidence hydrological conditions is also shown for two swamps within the study area. The area over the proposed Dendrobium Area 2 operation in the Southern Coalfield was chosen as a case study site, and high-resolution airborne laser scan data were acquired for the site from BHP Billiton.

INTRODUCTION

Ground movements attributable to coalmine subsidence have long been associated with effects to surface infrastructure, and more recently, natural features. Traditionally, the focus of subsidence impacts has been geared mainly towards man-made features such as transport infrastructure, buildings and installations, and pipelines (Kratzsch, 1983; Whittaker and Reddish, 1989). More recently, however, increasing attention has been given to natural features and impacts have been documented on features such as watercourses (Sidle et al., 2000), cliffs and steep slopes (Kay, 1991; Holla and Barclay, 2000), and aquifers (Booth, 2002; Dumpleton, 2002).

There is therefore a pressing need for coalmine operators to assess and understand potential subsidence impacts to natural features, both because of their social and environmental responsibilities but also because of legislative requirements. In NSW, these requirements are embodied in the Subsidence Management Plan (SMP) process, which came into effect during March 2004. SMPs are designed to satisfy increasing public and stakeholder concerns about environmental impacts associated with coalmine subsidence in NSW. The SMP process is comprehensive and includes requirements to assess potential impacts of subsidence to both natural and man-made infrastructure (Regan, 2003). Section 6 of the SMP Guidelines (NSW DPI, 2003) states that SMPs must “provide information that: (1) Characterises the nature, extent and magnitude of the expected subsidence impacts due to the proposed mining, and (2) Identifies priority risks, highlighting the expected subsidence impacts with high risk levels and/or potentially severe consequences.”.

One class of tools that offer significant potential for subsidence impact assessment is Geographic Information Systems (GIS). At its core, GIS exhibits the capacity to access, manipulate, analyse, and visualise spatial data. This then facilitates the derivation of new spatial data based on the attributes of existing, sometimes incongruent, spatial data sets (Figure 1). Although the large-scale use of GIS for subsidence impact assessment is yet to be realised, the successful application of GIS to complex spatial problems in other fields attests to its suitability for the task. Pertinent examples can be found, for example, in the work of Flentje et al. (in prep) or Chau et al. (2004) in the field of landslide management, Zhou et al. (2003) for regional land subsidence hazard mitigation (related to groundwater extraction), or Mansor et al. (2004) for natural risk management.

Central to the successful application of GIS to any problem is the acquisition of suitable data. While many datasets are required for comprehensive subsidence impact assessment, one dataset in particular that has broad application is topographic data. Digital topographic data can be sourced through a range of processes, including

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digitisation of topographic maps, aerial photogrammetry, terrestrial or airborne laser scanning, satellite-borne radar, and ground-based surveys including the use of Global Positioning Systems (GPS). Regardless of the source, once the topographic data have been interpolated to form a surface, usually through the creation of a digital elevation model (DEM), the data can readily be incorporated into GIS-based terrain analyses.

Perhaps one of the areas in which GIS-based analyses can be most readily applied, in terms of subsidence impact assessment and SMP preparation, is the “...full land use description and impact assessment, including the physical landforms and environment of the area...” upon which SMPs will be “built” (NSW DPI, 2003). On its own, the potential uses of topographic data include site characterization, feature identification, hydrological modelling, and more. When combined with other pertinent datasets in a GIS, topographic data can potentially be used to assess or predict potential impacts. The aim of this paper is to:

1. document some of the typical analyses that can be performed using GIS and suitable topographic data,
2. demonstrate how these fundamental analyses can provide a useful starting point for further assessments, and,
3. outline some of the main limitations and considerations that need to be understood for the efficient and effective use of the technology available.

![Fig. 1 - The overlay of multiple spatial layers in GIS](image)

The layers shown here include (from the top down) transportation, swamps, vegetation, the topographic wetness index, drainage lines, aerial imagery, a hillshade model of the topography, topographic contours, structural geology, regional geology, and mine plans. The ability of GIS to consider the attributes of multiple varied datasets at the same locations provides tremendous capacity for spatial modelling and analysis.
DATA AND OVERVIEW

This paper uses the proposed Dendrobium Area 2 mine, operated by BHP Billiton (BHPB), as the main case study for demonstrating the supporting role of GIS and topographic data in the subsidence impact assessment process. Beyond the use of a mine plan and subsidence predictions, the assessments shown here rely only on topographic data derived from an airborne laser scan survey conducted in February 2003.

The topographic data were interpolated into a digital elevation model (DEM) using triangulation and pixel resolution of 2 m. The choice of interpolation method and DEM resolution was based on the mean point spacing of the raw data and a comparison of interpolation methods as outlined in Palamara et al. (submitted). A 2.5D representation of the study area is shown in Figure 2 using a hillshade model of the topographic data.

SITE CHARACTERISATION

Many studies have demonstrated that topographic data and GIS can be used to perform a wide variety of terrain analyses, particularly with regard to hydrological analyses (Jensen, 1991; Moore et al., 1991; Wilson and Gallant, 2000). The site characterisation process can exploit these, and other, capabilities of GIS to identify areas that are likely to be susceptible and therefore warrant further assessment. In this instance, the examples presented here include the identification (i.e., mapping) of valley floor areas, major drainage lines, steep terrain (including cliffs), and erosion prone areas. Other natural features or attributes that can be readily characterised using topographic data and, in some cases supplementary data, include watersheds, flood prone areas, surface wetness/soil moisture and other important aspects.

VALLEY FLOOR AREAS

Valley floor areas that are subjected to mining-induced subsidence are potential sites for the occurrence of...
upsidence (Holla, 1997; Waddington and Kay, 2003a), which can result from valley closure. The identification of valley floor areas, which typically coincide with the location of rivers and creeks, can therefore be used to highlight areas of potential subsidence impacts.

Waddington and Kay (2003a) have indicated that ‘valley depth’ is one of the major factors controlling the magnitude of upsidence. Measuring valley depth through the use of digital topographic data is one method of both identifying low areas in the terrain (such as valley floors) and also quantifying the magnitude of relief. Figure 3 shows valley floor areas for the study site. In this instance, relative low points in the surface were identified by comparing the height at each location (i.e., each 2 m pixel) with the ‘regional’ height derived as an average of all heights in a 100 m radius surrounding each pixel. The choice of radius is arbitrary and can be easily modified. The most appropriate radius may be different for different types of analysis and is the subject of ongoing research.

Valley floor areas can subsequently be identified as areas in which the local height is less than the regional average height, and the magnitude of this difference provides an objective and accurate measure of valley depth. In Figure 3 relatively low areas (greater than 10 m height difference) are evident in many locations, but the maximum depth does not exceed 20 m. The deepest areas (15 m – 20 m depth) are situated in the large gully to the north of the study site (under the middle panel), and in the depression in the bottom left corner of the study site.

This method using GIS analysis can be considered superior to manual methods of measuring valley depth because it is independent of manual bias and inaccuracies, can be rapidly derived, is repeatable, and the results are available in digital format, which facilitates their possible use in modelling pursuits or empirical predictions. Furthermore, it does not suffer from the same problems that the manual method faces with undulating or irregular terrain (Waddington and Kay, 2003a).

DRAINAGE

Potential changes to watercourses in the form of surface fractures and water loss, ponding, and altered sedimentation regimes are a common concern in mine subsidence impact assessment. It is therefore critical to accurately identify drainage areas within the study site and to derive the appropriate attributes.

The continued and increasing use of GIS in hydrological studies has produced many techniques for the characterisation of regional hydrology, which naturally includes the identification of streams or drainage lines. Interpretation of watercourses and drainage lines can therefore be readily derived from topographic data using a variety of spatial algorithms and methods. Most methods generally involve the calculation of flow direction, upslope contributing area or flow accumulation, and surface slope and aspect, all of which is readily performed using only topographic data.

Figure 4 shows the prominent drainage lines within the study area. These will typically correspond to the location of known watercourses (subject to the accuracy of the DEM used to derive them). In Figure 4, as in most cases, the derivation of drainage lines based on topography will highlight all areas where surface water is likely to accumulate and flow, though in reality these areas do not always correspond to permanent watercourses. Nevertheless, the identification of drainage lines – whether permanent or ephemeral – is important given the potential impacts associated with both topographic changes and surface/sub-surface fracturing associated with subsidence. In Figure 4 it is evident that there are numerous drainage lines which lead to major watercourses, and eventually into the dams within catchment areas. Many of these features coincide with the proposed mining area or the ‘valley floor’ (upsidence) areas identified in Figure 3, and can therefore be flagged for further assessment.

The automated identification of drainage lines using digital topographic data offers numerous benefits over reliance on external drainage data, whether it is in digital or hardcopy format. The derived drainage data is likely to be more accurate and up-to-date than data extracted from topographic maps or the associated spatial databases, as shown in Figure 5. It is also likely to be more comprehensive since, as outlined previously, the technique will identify drainage areas that do not necessarily correspond to established watercourses, but instead reflect preferred or likely flow paths for accumulated surface water. Minor watercourses that are unlikely to be recorded on topographic maps or regional drainage databases will also be captured by this method. Also, by having the watercourse data in digital format it is possible to perform further analyses, such as stream order calculations or the comparison of pre- and post-subsidence profiles.
Pixels corresponding to valley floor areas were identified by comparing the local height with the average height over an area measuring 100 m in radius from each pixel. Depths of greater than 10 m were considered to correspond with valley floor areas. This objective, repeatable, and accurate technique rapidly identifies low areas in the terrain which, due to valley closure, may be prone to upsidence impact. Both the depth at which to assign the classification of “valley floor” (in this case, 10 m) and the radius of the “regional” average (100 m) are somewhat arbitrary and can be modified to suit the site.

**STEEP TERRAIN**

The identification of steep terrain, which can be susceptible to mine subsidence impacts due to its intrinsic instability and propensity for rock falls is relatively straightforward if high-resolution data are used. Figure 6 shows areas that may be susceptible to mine subsidence impacts based on the classification of surface slopes from the topographic data. To derive this image, slope values (in degrees) were calculated for the entire study area using the topographic data outlined earlier, and the resulting slope classes were classified using a simplified version of the slope classification table of McDonald et al. (1998). Relatively flat areas are shown in Figure 6 since these areas may be susceptible to flooding associated with subsidence-induced vertical movements. In this instance, though, the study site does not contain large expanses of flat ground. There are, however, many areas with very steep slopes which occur over the proposed mining area. In particular, there is a long (~2 km) extent of steep slopes situated directly over the proposed longwall panels.
As well as identifying and mapping these susceptible areas digitally, the topographic data can also be used to partially assess or predict subsidence impacts to these areas using either knowledge- or data-driven modelling, as outlined in Palamara et al. (2006) and Zahiri et al. (in press) and in a later section of this paper.

Fig. 4 - Major drainage lines for the study area
These lines were derived using the 2 m DEM and stream identification functions that are available in most GIS. The method allows for the objective and accurate identification of possible drainage line. The fact that each drainage line is available as a digital object in GIS facilities further analyses such as comparison of pre- and post-sub-subsidence height profiles, stream order assignment, and a determination of relative valley depth for each pixel along the drainage line.
Fig. 5 - Watercourses over a sample area from two different datasets
Note the inaccuracies in the 1:1,000,000 scale Geoscience Australia data set (red). The drainage network does not conform to the channel areas shown in the aerial photograph. The inaccuracies in these widely available data sets preclude them from being useful for subsidence impact assessments. The DEM-derived watercourse vectors conform to the channels somewhat more closely and were derived within ArcGIS™ using the freely available TauDEM extension (Tarboton, 2003). The sample shown here is situated near Appin, NSW, and the DEM-derived watercourses are based on a 5 m DEM provided by the University of Wollongong school of Earth and Environmental Sciences.

EROSION-PRONE AREAS

Mining subsidence has potential to either initiate or increase erosion through (1) the removal of vegetation due to water loss and drying, gas escape, rock falls or slope failure, or by (2) altering surface gradients and flow patterns. It is therefore worthwhile mapping erosion potential prior to mining, so that potential impacts can be quantified and if necessary mitigated. Numerous studies (Boggs et al., 2001; Pistocchi et al., 2002; Lufafa et al., 2003; Hoyos, 2005) have demonstrated that, to some extent, the quantification and mapping of erosion potential, and the identification or erosion hotspots, can be achieved using GIS and topographic data. For mine subsidence impact assessment, the aim of erosion mapping is to highlight areas that warrant further investigation, in the form of either field observations or more detailed modelling (perhaps incorporating further factors), and also locations that should be monitored due to erosion potential.

While more than one method exists, most GIS-based assessments of erosion potential employ the universal soil loss equation (USLE) or a revised form of this index (RUSLE), which is relevant for sheet and rill erosion. The index incorporates numerous parameters that are relevant to erosion, such as soil erodibility, slope and slope-length, runoff, surface and cover management, and conservation practices (Renard et al., 1994; Hoyos, 2005). Other methods also require a combination of parameters in order to model erosion potential well. However, it is also possible to model erosion potential using topographic factors alone, by using just the slope and slope-length factor (termed the LS factor) from the USLE or versions of the stream-power indices (Wilson and Gallant, 2000).
Susceptible terrain, such as level ground or very steep/precipitous slopes, was identified by classifying the surface slope values using the table of McDonald et al. (1998).

While erosion mapping based on topography alone will not present a complete picture of the actual erosion potential in the study site, it is useful for mine subsidence impact assessment because it will identify areas of potential risk that may warrant more in-depth modelling (incorporating all the suitable factors, not just topography) or field observations.

Figure 7 shows erosion potential as mapped by the LS factor (Desmet and Govers, 1996) from the (R)USLE for the study area. The LS factor is dimensionless it can be difficult to interpret; therefore in Figure 7 the results are classified by quantiles. This is somewhat a subjective representation, since the quantiles of the LS-factor will vary for each study site, though it succeeds in demonstrating which areas are most prone to sheet and rill erosion – namely the flanks of the ridge in the centre of the study area, and sections within the drainage gullies to the north and the south west of the area. As pointed out by Desmet and Govers (1996), the highest values occur primarily in areas with steep slopes and in zones of flow concentration.

This approach to the identification of erosion prone areas has the benefit of being easily and rapidly implemented and, as with the other analyses shown previously, the susceptible areas are mapped in digital format and can therefore be analysed and manipulated as required. However, it must be stressed that generic erosion mapping such as this is limited by many factors and should only be used as a guide on erosion potential when combined with assessment of other important aspects.
Fig. 7 - Erosion potential for the study site, based on the LS-factor.
The LS factor is the topographic component of the (R)USLE, and when calculated in the absence of other factors only provides a partial insight into potential erosion patterns. The LS factor (and the USLE) was designed for use of relatively flat slopes and is known to overestimate erosion potential on steep slopes. Despite its shortcomings, this figure demonstrates how the LS factor (or other erosion indices based on topography alone) can potentially be used to identify areas that may be susceptible to subsidence impacts. Note, in particular, the area (marked in red) at the head of the gully towards the north of the study site, as well as the confluence of high LS factor values within some of the drainage areas on the flanks of the central ridge.

ASSESSMENT

Topographic data can also feature heavily in the subsidence assessment stage, using either the natural features identified as part of the site characterisation process, or other forms of digital data. Various examples of GIS-based subsidence impact assessment, based primarily on the use of topographic data, are presented here including:

1. The measurement of stream profile changes using pre- and post-subsidence surfaces,
2. The assessment of subsidence impacts to cliffs and steep slopes (demonstrated for a different study site), and
3. Mapping of hydrological changes due to vertical movements, both in general and for two specific swamps within the study area.
STREAM PROFILES

Once an object has been recorded spatially within a GIS it is possible to extract attributes from other digital spatial data layers at that location for the purpose of analyses. For comparing changes to specific natural features due to subsidence it is possible to derive a post-subsidence surface (using subsidence predictions), and then compare the height changes for each feature.

Figure 8 shows the change in height profiles for two selected sections of drainage lines; both occur directly over the proposed mining area. The post-subsidence surface was derived using subsidence prediction data provided by Mine Subsidence Engineering Consultants (MSEC) Pty Ltd. For both profiles the areas of greatest change are readily evident, and in the second profile the location of a probable natural (i.e. pre-subsidence) pond is evident. The comparison of pre- and post-subsidence profiles could help with the identification of areas in which subsidence might result in flow reversal, ponding, or fracturing.

CLIFFS AND STEEP SLOPES

The following section describes the assessment of potential subsidence impacts to cliffs and steep slopes using two fundamental approaches, both of which can be readily performed within a GIS if the suitable supplementary data are available; a more detailed description of these approaches can be found in Palamara et al. (2006).

In the first instance, a knowledge-driven approach can be adopted. This requires an in-depth understanding of the pertinent factors which influence the magnitude or severity of the subsidence impact. The GIS is then used, where possible, to quantify each factor in order to provide a final assessment of possible subsidence impacts. For example, Waddington and Kay (2003b), in their handbook, outline a knowledge-based assessment system for cliffs. Many of the factors outlined in their assessment system, such as the ‘extent of mining’, and ‘degree of public exposure,’ can be either derived directly within a GIS (using topographic and other data), or incorporated into a GIS. A type of ‘multicriteria’ analysis, which is commonly implementing using GISs for site selection and suitability analyses (Malczewski, 2004) can then be performed, where each factor is derived for each point in the study area, weighted, and summed according to the specifications of the assessment system.

An alternative, which does not require an in-depth understanding of the relevant factors, is data-driven modelling. This can be accomplished through a variety of methods, but in all cases require a suitable database of mapped subsidence impacts, so that the data-driven model can empirically identify and weight the relevant factors. Unlike knowledge-based assessment, which generally provides a classification of the system under examination, data-driven modelling will typically provide a ‘probability’ value based on the confluence of the relevant factors. Further information on this case study is available in Zahiri et al. (in press), which uses a database of rockfalls associated with the nearby Tower Colliery workings for the modelling experiment.

Although vastly different in their output and implementation, both methods can be readily performed using GIS and rely heavily on topographic data. Furthermore, they can be implemented for a variety of natural features, not only steep slopes and cliffs.

HYDROLOGICAL CHANGES

Subsidence impacts can be attributed to two main mechanisms – topographic changes associated mainly with vertical movements and surface and subsurface fracturing associated with zones of tension and compression. The majority of subsidence impacts can be attributed to this later mechanism, which is unfortunately relatively difficult to model and predict. Conversely, because accurate subsidence predictions are readily available, subsidence impacts associated directly with topographic changes are relatively easy to model.

For example, the plethora of hydrological indices that can be readily calculated using GIS and topographic data can be used to evaluate subsidence impacts relating to topographic changes alone by the simple comparison of pre- and post-subsidence surfaces. The same parameters outlined in the site characterisation section, and more, can be evaluated for subsidence-induced changes.

A comparison of these parameters was performed for two swamps which occur within the study area. Swamps are known to be sensitive to environmental changes and are potentially important in subsidence impact assessment (Horsley and Brassington, 2004; Horsley, 2003). Pre- and post-subsidence values for some hydrological attributes
were extracted for the two swamps which occur in the study area (Figure 9) and compared to determine how the swamps may be influenced by subsidence (Table 1). The swamps were mapped digitally, in the first instance, by Horsley (2003) and have been adjusted in this exercise based on the visual inspection of high-resolution (20 cm) aerial photography.

Table 1 shows that changes in hydrological parameters, associated with altered topography due to predicted subsidence, are minimal. For Swamp 1, which is situated directly over the proposed mining area, the most noticeable change is a decrease in catchment area. The term ‘catchment area’, for a GIS or terrain analysis perspective, is tantamount to the flow accumulation for a cell, and refers to the count of cells which flow into a particular cell. If runoff values are known (in this case they are not) they can also be included to derive a more accurate estimate of the amount of water flowing into a particular cell. In this case, subsidence is predicted to decrease the catchment area for Swamp 1, on average, by 14%. The impact of such a change on swamp health is not contemplated here, but is worth noting for subsidence impact assessment purposes. When other parameters are considered, this change in catchment area is manifested as a slight decrease in the mean and maximum wetness index within the swamp area, and also a slight decrease in the value of the corresponding erosion indices. Swamp 2, which is not situated directly over the proposed longwalls, is expected to undergo only a slight decrease in catchment area, and therefore not experience significant changes due to topographic adjustments.

![Fig. 8 - Pre- and post-subsidence profiles for selected drainage profiles.](image)

Subsidence prediction data (provided by MSEC Pty Ltd) was used with the original topographic data to produce a post-subsidence surface. The digital mapping of drainage lines (outlined earlier) allows for attributes from other spatial layers (in this case, pre- and post-subsidence surfaces) to be analysed for each feature. Here the changes in drainage line profiles can be seen for the selected watercourses. This approach, which is easily facilitated using GIS, can potentially be used to identify areas of ponding or changes in flow direction for selected watercourses.
The swamps were originally mapped and identified by Horsley (2003). The swamp boundaries have been modified here based on the visual inspection of high-resolution aerial photography. Swamps are potentially susceptible to mine subsidence because of their strong dependence of hydrological conditions. An example evaluation of these swamps, in light of the topographic changes associated with the predicted subsidence, is given in the text.

Fig. 9 - The location of swamps within the study area.
Table 1 - A comparison of selected pre- and post-subsidence hydrological parameters for swamps within the study area.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Swamp 1</th>
<th>Swamp 2</th>
<th>Change</th>
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<tbody>
<tr>
<td></td>
<td>Pre</td>
<td>Post</td>
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<tr>
<td>Subsidence (mean)</td>
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<tr>
<td>Subsidence (max.)</td>
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<td>1.7 m</td>
<td>-</td>
</tr>
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<td>Wetness Index (mean)</td>
<td>8.3</td>
<td>8.1</td>
<td>-0.2</td>
</tr>
<tr>
<td>Wetness Index (max.)</td>
<td>10.2</td>
<td>11.6</td>
<td>1.4</td>
</tr>
<tr>
<td>LS factor (mean)</td>
<td>6.7</td>
<td>6.5</td>
<td>-0.2</td>
</tr>
<tr>
<td>LS factor (max.)</td>
<td>26.9</td>
<td>29.7</td>
<td>2.8</td>
</tr>
<tr>
<td>Stream Power (mean)</td>
<td>69</td>
<td>60</td>
<td>-9</td>
</tr>
<tr>
<td>Stream Power (max.)</td>
<td>552</td>
<td>510</td>
<td>-42</td>
</tr>
<tr>
<td>Slope (mean)</td>
<td>6.4°</td>
<td>6.5°</td>
<td>0.1°</td>
</tr>
<tr>
<td>Slope (max.)</td>
<td>11.9°</td>
<td>13.4°</td>
<td>1.5°</td>
</tr>
<tr>
<td>Catchment Area (mean)</td>
<td>1,150 cells</td>
<td>982 cells</td>
<td>-168 cells (-14%)</td>
</tr>
<tr>
<td>Catchment Area (max.)</td>
<td>5,465 cells</td>
<td>4,575 cells</td>
<td>-890 cells (-16%)</td>
</tr>
</tbody>
</table>

Refer to Figure 9 for the location of the swamps. These changes were calculated using hydrological analysis functions intrinsic to the SAGA 2.0b GIS and topographic data. The largest change is evident in the post-subsidence decrease in catchment area for Swamp 1. This means that, based on the predicted subsidence, the area from which surface water will collect and flow into Swamp 1 will decrease due to topographic changes. Spatially variable runoff has not been considered in this analysis. It is not clear what effect, if any, this will have on the swamp, since channel processes may prove indifferent to this change. Even though the changes shown here are likely to be of little consequence, they demonstrate the capacity of topographic data, when coupled with GIS, to provide for fundamental subsidence impact assessment.

**DISCUSSION**

The aforementioned analyses demonstrate the capability of GIS to perform ‘accurate’ and rapid site characterisation and preliminary identification of susceptible features for subsidence impact assessment. The results of these analyses are summarised in Figure 10, which shows some of the natural features in the study area that are potential susceptible to subsidence impacts, which include:
1. the swamp situated directly over the proposed mine area,
2. some ‘valley bottom’ areas that are greater than 15 m lower than the surrounding average heights,
3. the steep slopes, particularly those near the centre of the proposed longwalls, and
4. large areas with relatively high erosion potential.

The benefits of this approach to subsidence impact assessment, which entails the use of digital topographic data and GIS, are numerous:

1. The results shown here were accomplished without the need for fieldwork,
2. were rapidly derived,
3. are readily repeatable using different parameters if necessary, and
4. the product is in digital format and can therefore be easily distributed, visualised, and manipulated.

The map shown in Figure 10 could serve mainly as a starting point for further, more detailed analyses. When supported by established empirical methods, knowledge-based assessment systems, or databases of mapped impacts suitable for data-driven mining, the analyses presented here can be extended from simple mapping exercises to predictive projects with relative ease.

There are, however, some important considerations associated with this approach. For example, the analyses presented within this paper have been derived using a topographic surface generated from airborne laser scan data. Not all topographic data share the high vertical accuracy and horizontal resolution of airborne laser scans. The horizontal resolution of topographic data will be one of the controlling factors which determine what type of terrain analyses can be undertaken and how reliable the results will be. As resolution decreases (that is, pixel size becomes larger) steep slopes and narrow features become more difficult to resolve. Slope-based classification, as shown earlier for the mapping of steep slopes and precipitous areas, becomes difficult. Even at 5 m resolution the ability to identify cliffed areas based on slope is severely limited. The visual inspection of hillshade or 2.5D representations will remain a valuable tool for the estimation of possible cliff areas, even with lower resolution data, but field mapping will more than likely also be required. Similarly, the identification of drainage lines will also become difficult as resolution decreases, and other methods such as field observations and aerial photograph interpretation may be needed.

Other considerations include the need for specialist software and comprehensive databases of subsidence impacts. Without these critical inputs, the scope for actual assessments using digital data and GIS is somewhat limited. At this stage, output such as that shown in Figure 10 cannot be considered as ‘susceptibility maps’ because it is not clear how the susceptibility of each feature should be assessed. As suitable mapped impacts become more available it may be possible to develop risk assessment maps that consider frequency and consequence of the subsidence impacts. When that occurs, GIS will become a critical tool not only for the presentation and assimilation of data for subsidence impact assessment and SMP purposes, but also to meet the requirement outlined in Harvey (2003), which is to “…be able to categorically determine the degree of impact a particular amount of subsidence will have on a surface feature…”.
Fig. 10 - An overview of the site characterisation for the study area
The figure summarises the characterisation process outlined in the text and displays some of the features that can be considered most susceptible to subsidence impacts. The analyses required to derive this figure are relatively straightforward and require only topographic data. The figure demonstrates the capability of GIS to act as a starting point for subsidence impact assessment by accurately mapping, and in some cases assessing, potentially susceptible features.

ACKNOWLEDGEMENTS

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REFERENCES


Palamara, D, Flentje, P, Baafi, E, and Brassington, G (submitted) Using airborne laser scan data for coalmine subsidence mapping.


A CASE STUDY ON LONGWALL MINING UNDER THE TIDAL WATERS OF LAKE MACQUARIE

Gang Li¹, Ian Forster², Matthew Fellowes³ and Andrew Myors⁴

ABSTRACT: This article presents a case study of longwall mining under the tidal waters of Lake Macquarie, south of Newcastle, NSW. The mining operation took place at the Wyee Colliery from 1998 to 2002, without any incidence of ingress of water from the lake. This paper describes i) the design strategy used for a viable and robust mine layout that minimises the risk of water ingress ii) observations made during the operational phase and iii) a practical tool for designing underground coal mines under surface water bodies.

INTRODUCTION

This article presents a case study of longwall mining under the tidal waters of Lake Macquarie, south of Newcastle, NSW. The mining operation took place at the Wyee Colliery from 1998 to 2002, without any incidence of ingress of water from the lake or any abnormal underground water makes.

The water in a tidal lake is controlled by the sea to which the lake is connected. Therefore, for mine design and operation purposes, the task was to manage the risk of potential hydraulic connection between the underground mine workings and an inexhaustible body of surface water.

At the design stage, the most significant challenges were:

• The limited thickness of solid overburden strata (151 m to 178 m) relative to the requirement of the Wardell Guidelines (1975). Therefore, there was a need to understand the nature, magnitude and distribution of mining-induced surface and sub-surface ground deformations/fractures, and

• Part of the planned mining was in geologically disturbed areas. It was recognised at the design stage that the most significant unknown was whether or not the geological structures would act as conduits under the influence of mining-induced stresses and strains.

This paper describes the design strategy used for a viable and robust mine layout in the context of managing water ingress risk. The strategy comprised primarily:

• Summary of past experience of mining under surface water bodies;
• Characterisation of site-specific conditions relevant to the subject;
• Establishment of site-specific geotechnical and hydrogeological models for mining-induced surface and sub-surface deformations;
• Assessment of the interactions between geological structures and mining-induced surface and sub-surface deformations, and
• Development of mine design measures to minimise any potential impacts of the identified uncertainties and potential variations in site conditions.

This article also comments on the implemented mine layout based on the observations made during the operational phase and presents a practical tool for designing underground coal mines under surface or overlying water bodies.

PAST EXPERIENCE OF MINING UNDER SURFACE OR OVERLYING WATER BODIES

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The lessons from studies of over 30 previous inrush/inflow cases reported in the literature (e.g. Singh 1986) and unpublished reports are that such incidences occurred due primarily to:

- Ambitious layout or excessive resource recovery relative to the thickness and competency of strata between the underground workings and the overlying water bodies. Inadequate solid rock cover was primarily responsible for most of the reported inrush/inflow events;
- Geological structures in combination with incompetent/permeable overburden strata, which were responsible for some apparently “abnormal” inrush or inflow incidences;
- Lack of understanding of site conditions or the nature, magnitude and distribution of mining-induced ground deformations/fractures, and
- Variations in site conditions and uncertainties, which were not adequately addressed in the mine designs and management processes.

These lessons influenced the development of the mine design strategies used for the project.

**MINING DATA AND SITE CONDITIONS**

**Mining data**

Figure 1 shows the layout of Longwalls 17 to 23 in Wyee Colliery, all of which are sub-critical panels (Table 1) within the Fassifern Seam. As shown in Table 1, the cover depth (from the surface of lake bed sediment to the top of the Fassifern Seam) ranges from 161 m to 197 m, whereas the thickness of rock cover (i.e. overburden strata excluding lake bed sediment) varies from 151 m to 178 m.

There are no existing workings at the subject site, with the exception of a small area of first workings in the overlying Great Northern Seam.

**Table 1 Summary Data - Mining under Lake Macquarie at Wyee Colliery**

<table>
<thead>
<tr>
<th>Panel Name</th>
<th>LWs 17 to 19</th>
<th>LW20</th>
<th>LW21</th>
<th>LWs 22 &amp; 23</th>
</tr>
</thead>
<tbody>
<tr>
<td>Void Width (m)</td>
<td>130</td>
<td>140</td>
<td>140</td>
<td>150</td>
</tr>
<tr>
<td>Mining Height (m)</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
<td>3.2</td>
</tr>
<tr>
<td>Chain Pillar Height (m)</td>
<td>3.2</td>
<td>n/a</td>
<td>n/a</td>
<td>3.2</td>
</tr>
<tr>
<td>Chain Pillar Width (m)</td>
<td>45</td>
<td>n/a</td>
<td>n/a</td>
<td>45</td>
</tr>
<tr>
<td>Chain Pillar Length (m)</td>
<td>51-95</td>
<td>n/a</td>
<td>n/a</td>
<td>145</td>
</tr>
<tr>
<td>Cover Depth (m)</td>
<td>162-174</td>
<td>175-181</td>
<td>161-185</td>
<td>179-197</td>
</tr>
<tr>
<td>Rock Cover (m)</td>
<td>151-159</td>
<td>158-162</td>
<td>157-173</td>
<td>162-178</td>
</tr>
<tr>
<td>Lake Bed Sediment (m)</td>
<td>11-21</td>
<td>17-23</td>
<td>11-23</td>
<td>18-26</td>
</tr>
<tr>
<td>Recorded Smax* (m)</td>
<td>0.65</td>
<td>0.4</td>
<td>0.45</td>
<td></td>
</tr>
</tbody>
</table>

* Smax = Maximum subsidence

**The lake bed sediment**

As shown in Table 1, the thickness of the lake bed sediment ranges from 11m to 26m in the subject area. The unconsolidated sediment consists generally of clay or silty clay with lenticular bodies of silty sand, sand and gravel. The materials have the potential to seal fractures and other mining-induced voids.

**In-situ stress**

Unusually high horizontal stress (about 5 to 7 times of the vertical) was measured at the adjacent Kangy Angy road cutting (Chappell, et al, 1984) and from a number of mine workings at depths less than 200m in this region (DMR, 1997).
According to the documented in-situ stress regime in the Newcastle-Gosford Region (DMR, 1997, Enever et al., 1998, Enever & Clark, 1998, Zhang et al., 1996), it is assessed that the in-situ major horizontal (compressive) stress is oriented NNE at Wyee, which would be sub-perpendicular to the dominant geological structures and would therefore act against the opening of these structures. Consequently, the high horizontal stress field should generally be beneficial from the viewpoint of controlling water ingress risk.

**Stratigraphy and strata conditions**

The Fassifern Seam is stratigraphically located in the upper section of the Late Permian Newcastle Coal Measures overlain by the Triassic Narrabeen Group. Figure 2 shows the generalised stratigraphy of the subject area to facilitate the discussions to be presented in this paper.

As illustrated in Figure 2, the overburden strata in the subject area are characterised by the dominant occurrence of massive conglomerates, separated by coal seams and tuffaceous rock units (e.g. Mannering Park Tuff and Awaba Tuff). Fresh conglomerate strata in the region have both high UCS strength and fracture resistance (Moelle et al., 1996), whereas the tuffaceous rocks, where they are degraded or mechanically weak, often behave plastically with a potential to seal fractures and other mining-induced voids.

**Massive conglomerate units**

A massive unit is geotechnically defined as a stratum that is substantially free from horizontal defects so that it may behave mechanically as a beam. The horizontal defects, which are the key features used for the identification of the massive units, may include bedding partings, erosional surfaces, horizontal structural discontinuities and layers of fine-grained sediments, such as mudstone, siltstone or fine-grained sandstone, etc.

The assessment of massive conglomerate units was aimed at determining the following:

- The location, thickness and mechanical properties of the massive units, and
- The distance between the top of the Fassifern Seam and bottom of the identified massive units.

The assessment of massive units was made based on geological logs from 25 boreholes surrounding the subject area. The stratigraphic units studied included the Karingal, Teralba, Karignan and Munmorah Conglomerates. Their locations relative to the Fassifern Seam are illustrated in Figure 2.
For reasons of conservatism (Sheehan, et al., 1997), the identification of the thickness of the massive units requires the assessment towards the lowest possible limit. This was done by using identifiable horizontal geological defects and any fine-grained sedimentary rocks as dividers, irrespective of their thickness. Thin medium-grained sandstone layers may be included as part of the massive units, judged by the overall pattern of the massive unit development. In addition, the weathered section of the uppermost Munmorah Conglomerate was excluded. The results of the assessment are presented later. The values of UCS, Young’s Modulus and frictional angle of the massive units were 65 MPa, 18500 MPa, and 35°, respectively, based on test results.

Geological structures

The geological setting of the Newcastle-Gosford Region is determined by the geometry and depositional history of the northern fringe of the Sydney Basin. It has been affected by post-depositional ductile and brittle tectonic deformational events. These events included primarily the creation of several regional folds, and subsequent faulting and igneous dyke emplacements, observable at the subject site.

Based on the results of surface/underground mapping, geophysical surveying and bore logging of the subject site and its surroundings, the following observations can be made:

- The majority of the structures in the subject area consist of NW trending, near vertical normal faults and their associated dykes. These structures were generally dry and tight and significant disturbances to the surrounding strata were not observed;
- Based on the Early Tertiary age of the geological structures in the subject area, they have a potential to affect both the Permian Newcastle Coal Measures and the overlying Triassic Narrabeen Group, through the full overburden height. It follows that the structures’ potential to form conduits between the lake and the underground workings must be critically assessed and managed;
- Although the longwall panels were positioned at the design stage to avoid the known large structures using the best information available at the time, variations in structural conditions must be expected and managed, and
- The longwall were located in areas with different structural conditions and with different orientations in relation to the dominant NW trending structures. In particular, Longwalls 19 and 21 are located in a geologically disturbed area (Figure 3). Therefore, the mine design (and the risk management process) needed to be robust enough to deal with any significant variations and uncertainties.
The geotechnical and hydrogeological assessment was aimed at establishing site-specific models that provide:

- A guide for designing the subject longwalls against water ingress risk, and
- A basis for effective communication of risks among all involved in the management of the risks.

**Surface and sub-surface deformations**

The transmission of water through the overburden strata may take place via a number of mechanisms such as i) inter-granular porosity, ii) mining-induced voids, fractures and strata dilation/bed separations and iii) structural discontinuities/geological defects.

Forster (1995) presented a hydrogeological model for the Central Coast Region, which divides the overburden into four different zones with different ground deformation characteristics, as shown in Figure 4. To provide a practical tool for designing subaqueous mining, this model was calibrated against the hydrogeological data relevant to super-critical panels in the Central Coast Region including the Wyee Colliery (Forster, 1995). In contrast to the required rock cover of 60t ($t = $ extraction thickness) by the Wardell Guidelines (1975), the Forster Model suggested a reduced rock cover of 45t plus 10 m subject to further site-specific verification (Forster, 1995).

This article presents an enhancement to the Forster Model used during the project. As compared with the original Model, as shown in Figure 4 (Forster, 1995), the modifications were made to:

- Combine the "Caved Zone" and “Fractured Zone” into a single zone termed the “Dewatered Zone” located immediately above the extracted coal, and
- Introduce the findings of a number of recent studies dealing with the nature and distribution of the “Dewatered Zone”.

Consequently, the enhanced Forster Model divides the overburden into three major zones, namely, the Surface Zone, Constrained Zone and Dewatered Zone, as discussed below.
The Dewatered Zone

The Dewatered Zone, by definition as discussed above, provides an effective conduit between the underground workings and any overlying water bodies, which are either directly intersected by or hydraulically connected to this Zone. It follows that the concept of the Dewatered Zone highlights the critical importance of the thickness and integrity of the overlying Constrained Zone that is to function as a barrier to inflows from any overlying water bodies.

Studies on sub-surface deformations by Whittaker and Reddish (1989) show that the Dewatered Zone (as defined here) is bounded by hydraulically connected tensile fractures over the ribs, capable of transmitting large amounts of water to underground workings.

It is assessed that the Dewatered Zone above the extracted void is likely to develop a dome-shaped geometry as shown in Figure 5. This important assessment is based on a comprehensive investigation into sub-surface deformations involving goaf drilling, geophysical testing and several years’ of gateroad mapping in strata about 35 to 40 m above the old Liddell longwall workings at Cumnock Colliery (Li & Cairns, 2000). Further evidence can be found in documented investigations on sub-surface deformations (e.g. Colwell 1993, Kelly et al, 1998, Mills, 1998).

The height of the Dewatered Zone was assessed to be up to 33 times the mining height for the subject site. This was the estimated maximum height of the Dewatered Zone based on super-critical panels in the Central Coast Region (Forster 1995). Subsequent verification tests for the present project by Forster (1998) at Wyee Colliery produced data to further support this assessment.

The Surface Zone

For descriptions of this deformation zone, reference is made to Figure 4 and the original Forster Model (1995). The significance of the Surface Zone has been demonstrated by a case study reported by Singh (1986) at the North Derbyshire Colliery, where the recharge of the “bed separation zone” through the fractures/joints in the surface zone caused an inundation event.

To assess the potential impacts of the Surface Zone on water ingress, the depth of tensile cracks was assessed during the project based on the test results and principles of rock fracture mechanics (Li & Moelle, 1993). It was estimated that the depth of any surface tensile cracks for the present case would be unlikely to reach 10 m from the ground surface.

The Constrained Zone

Again, for descriptions of this zone, reference is made to Figure 4 and the original Forster Model (1995). From the results reported by Holla (1986 & 1990) and Luo and Peng (2000), it can be seen that shear dilation, bed separations and the resulting changes in horizontal permeability can affect large portion of the overburden, however, with reducing intensity away from the extracted horizon. It is important to note that although some mining-induced changes in permeability may still take place within the Constrained Zone, it may still be designed to function adequately as a barrier to prevent hydraulic connections between the overlying water bodies and the Dewatered Zone. According to the studies by Forster (1995, 1998), the thickness of the Constrained Zone, as such a barrier, should be equal to or greater than 12 times the mining height assuming no significant geological structures within the Zone.

As discussed above, the level of water ingress risk at the subject site is critically determined by the thickness and integrity of the Constrained Zone and is also related to the occurrence of faulting and other geological discontinuities.
Interactions between geological structures and mining-induced ground deformations

It was assessed that the near vertical faults/dykes would not significantly change the height of the Dewatered Zone above an isolated panel. It follows that for a single sub-critical panel, the use of a Dewatered Zone height of 33t (t = extraction height), which is based on super-critical conditions, will provide a degree of conservatism as part of the risk management strategies, when applied to the sub-critical panels beneath the lake in this case.

Although the shape of the Dewatered Zone may be altered due to the presence of geological structures, the extent of such alteration should be limited since the geological structures are near vertical. However, to minimise the potential for interactions between the Dewatered Zones above the individual panels, it was decided to adopt a specific design measure so that the Dewatered Zones above adjacent longwalls could be effectively separated. This was achieved by using appropriate chain pillar widths, as illustrated in Figure 5. Despite the above-mentioned design measures, hydraulic connection between the lake and the mine workings would still be possible if the near vertical faults/dykes were activated/connected by ground deformations within the Surface and Constrained Zones. The spanning capacity of the massive units within the overburden was utilised to manage this risk.

Spanning massive unit

In the context of managing water inflow risks, the designed functions of the spanning massive units were to minimise surface tensile strains, the development of mining-induced fractures and, importantly, the interactions between the geological structures and the ground deformations.

The assessment of the spanning capacity of the massive units was made based on the Voussoir Beam Theory (Brady & Brown, 1985).
Notes:

According to a case study at Cumnock Colliery by Li and Cairns (2000), the average value of \( \alpha \) was approximately 15 degrees (actual observations vary between 0 and 22 degrees).

For conglomerate overburden, \( \alpha \) angle is likely to be higher according to Whittaker et al (1989).

Sources of the above reference are listed in the paper.

Fig. 5 - Dome-shaped Dewatered Zones (The Dewatered Zone consists of the Caved Zone and Fractured Zone as Shown in Figure 4 above)

- The spanning distance is assumed to be the full panel width for all massive units in the overburden despite their different locations in relation to the Fassifern Seam. For the massive units in the Karignan and Munmorah Conglomerates (Figure 2), the use of the full panel width as the spanning distance is conservative.
- The surcharge on the beam is assumed to be the dead weight of the full overburden above the massive unit, including the weight of the beam itself. Again, this is a conservative assumption.
- The design factor against compressive failure of the conglomerate beam was selected as 1.2. It is important to note that the value of the design factor was selected in conjunction with the above conservative assumptions and by referring to the results of previous unpublished and published case studies in a similar geotechnical environment (e.g. Frith & Creech, 1997).

Based on the above assumptions and the mechanical properties of the conglomerates as quoted earlier, the conglomerate beam thickness required for spanning across various void widths from 130m to 170m was determined and is presented in Figure 6.

A comparison of the required beam thickness (Figure 6) for the panel widths shown in Table 1 with the assessed massive unit thicknesses has resulted in a plot (Figure 7) showing the indicative distribution of the spanning strata over the subject area and its surroundings. The common occurrence of spanning massive units in the subject area, as indicated by Figure 7, was confirmed by reduced subsidence in the subject area after the completion of mining.

Fig. 6 - Required Conglomerate Beam Thickness to Span across Mine Openings
Potential variations and uncertainties

The geotechnical assessment identified a number of potential variations and uncertainties requiring consideration:

- Unexpected variations in thickness of solid overburden strata;
- Unexpected significant geological structures, outside the requirements of the Wardell Guidelines (1975);
- Variations in mechanical properties/integrity of the Constrained Zone (i.e. the barrier to potential water ingress);
- Variations in thickness/mechanical properties of the massive units;
- The capacity of massive units to span across multiple panels even though they are sub-critical panels separated by large chain pillars, and
- Effects of geological structures affecting the stability of chain pillars and spanning capacity of massive units.

MINE DESIGN STRATEGIES

Principal design strategy

Considering the severity of consequences of a potential inrush incidence, the principal design strategy used in the project aimed to ensure minimal residual risks. The implemented measures were:

- To maintain adequate thickness and mechanical integrity of the Constrained Zone to form an impermeable barrier between the lake and the Dewatered Zone. This was primarily achieved by ensuring the minimal required thickness of rock cover in accordance with the Forster Model (i.e. the
thickness of rock cover $\geq 45 + 10$ m, with $t$ being the extraction height), rather than the Wardell Guidelines. By selecting a mining height of 3.2m, this requirement was principally met, and

- To develop a robust mine layout design, supported by a risk management system during the operational phase, to manage the risks of any re-activated geological structures, uncertainties or unexpected variations as discussed above, while maintaining a viable mining operation. This was achieved by building into the design a number of additional measures discussed below.

Additional design measures

The additional measures included:

- Appropriate panel width and stable chain pillars (Table 1) to ensure spanning of the massive strata across the longwall panels. The functions of the spanning strata have been discussed above;
- Separation of individual sub-critical panels by adequately sized chain pillars to limit panel interactions as well as the height/extent of Dewatered Zones, and
- A number of conservative assumptions used in the geotechnical models, as discussed above.

The potentially beneficial sealing effects of the lake-bottom deposits and tuffaceous strata and the effects of high in-situ stresses preventing opening of the geological structures were not considered when defining the layout parameters (Table 1), as an additional measure for an appropriate safety margin.

In summary, the key design issues were to use sub-critical panels, an appropriate mining height and suitable chain pillar sizes so that the dome-shaped Dewatered Zones above each of the extracted panels could be sufficiently separated from each other and be limited to a maximum height of 33 times the mining height with a degree of conservatism.

Figure 8 is an illustration of the major elements of the design process against water ingress risks, which contributed to the final mine layout shown in Figure 1 and Table 1.

Observations

The main observations made during the extraction of the subject panels are summarised as follows:

- Irrespective of the differences in the degree of structural disturbance, orientation of panels in relation to the dominant NW trending geological structures and any other variations in strata conditions, ingress of water from the lake or abnormal water makes were never detected during the extraction of the subject panels;
- Surface subsidence over the subject panels was surveyed using the baythymetric method. Although the accuracy of this method is much lower than that of the conventional methods used for land surveys, the magnitude of the recorded subsidence (Table 1) provides a clear indication that there was reduced subsidence across all subject panels confirming the existence of spanning massive strata within the overburden;

Fig. 8 - Design Process against Water Ingress Risk
• Longwalls 19 and 21 are located in a geologically disturbed area. Several normal faults are parallel and within the two panels with throws up to 1.2 m. These faults remained generally dry during the extraction and the test results of seepage from the faults showed that the water was not from the lake, and

• The development of the tailgate for Longwall 21 exposed an unexpected fault zone with a throw greater than 3m, probably up to 3.5 m. Subsequent exploration at other locations along the gateroad further confirmed the existence of this parallel fault in close proximity to Longwall 21. The structural conditions were outside the requirements of the Wardell Guidelines (1975), which recommend no extraction under tidal waters within 50 m of a fault with displacement greater than 3 m or a dyke of thickness greater than 6 m. The subsequent reviews conducted by the management team suggested that there would be minimal interactions between this fault and the dome-shaped Dewatered Zone, as illustrated in Figure 9. Longwall mining proceeded with the support of various actions documented in the risk management system developed and implemented by Wyee Colliery. Again, no ingress of lake water or abnormal water makes were observed during the extraction of this panel. Note that Longwall 21, like the adjacent Longwalls 17 to 19, was retreating down-dip meaning that any abnormal water makes would be easily detectable.

![Diagram of a Normal Fault with Throws up to 3.5m and Dip Angles 37 to 65 Deg.](image)

Fig. 9 - Dewatered Zone in Relation to the Unexpected Fault in TG21

Comments

The Wardell Guidelines (1975) are likely to be overly conservative at the subject site and possibly for other Central Coast areas with similar geotechnical and hydrogeological conditions.

The enhanced Forster Model discussed in this paper provided a practical tool for the design of the subject longwalls against water ingress risks. The design tool was developed based on the understanding of surface and sub-surface ground deformations at the subject site. Importantly, this understanding has facilitated the development of the mining system’s capability in dealing with unexpected geological disturbances as well as variations in site conditions, as discussed above.
It follows that the application of the Forster Model, being the original (Forster, 1995) or the enhanced version as presented in this paper, to other areas may be possible provided that there are adequate investigations and understanding of the site conditions and ground deformation characteristics. As demonstrated by this case study, such site-specific investigations and understanding as well as any necessary modifications to the Model are critical for the successful application of this design tool.

It may be argued that the implemented mine design is conservative and there could be room for better resource recovery for the present case. While it is worthwhile to further improve our understanding of the ground deformations caused by longwall mining in order to achieve optimal resource recovery, the required safety margin, which can be systematically developed through various measures as demonstrated by the present case (e.g. by limiting the panel widths to ensure spanning of the massive strata), needs to be critically considered for any particular mine sites.

REFERENCES


DRM (NSW Department of Mineral Resources), 1997. Sydney Basin Stress Map


Holla, L and Buizen, M, 1990. The strata movement and changes in bulk permeability due to longwall mining under an old goaf. The Coal Journal, No. 28, pp17-27


MONITORING OF SUBSIDENCE MOVEMENTS AT MAJOR INFRASTRUCTURE

James Barbato\textsuperscript{1}, Daryl Kay\textsuperscript{1}, Hank Pinkster\textsuperscript{2}, and Ben de Somer\textsuperscript{3}

\textit{ABSTRACT:} Time based mine subsidence predictions provide a valuable tool, as part of an overall management strategy to protect infrastructure, which involves making subsidence predictions at set increments of longwall travel. The predictions can be presented as a series of subsidence contours or profiles, and can be animated to show the progression of the subsidence travelling wave. The observed movements at major items of infrastructure can then be compared to the predicted movements at any time throughout the mining period. The challenges in providing time based predictions are discussed. Two examples are provided: the Main Southern Railway at Tahmoor Colliery and the gas and water pipelines across an unnamed Creek at West Cliff Colliery. Both examples show that time based predictions can provide a useful tool as part of an overall management strategy where major items of infrastructure are mined beneath. Time based predictions can be readily provided for any major item of infrastructure using current methods of subsidence prediction.

INTRODUCTION

Major items of infrastructure have been mined beneath, are currently being mined beneath, and are proposed to be mined beneath within the NSW Coalfields. The major items of infrastructure include freeways, major roads, railways, gas pipelines, water pipelines, electrical services and telecommunication services.

Time based mine subsidence predictions provide a valuable tool, as part of an overall management strategy to protect infrastructure, which involves making subsidence predictions at set increments of longwall travel. The predictions can be presented as a series of subsidence contours or profiles, and can be animated to show the progression of the subsidence travelling wave. The observed movements at major items of infrastructure can then be compared to the predicted movements at any time throughout the mining period.

Time based predictions can be used to provide trigger levels for management strategies when observed movements exceed predicted movements. They can also be used as a guide for the early detection of irregular subsidence movements.

ADVANCEMENTS IN TIME BASED PREDICTIONS AND MONITORING

In the past, one difficulty with providing time based predictions was the amount of calculation required to be undertaken. However, with the advancement and improvements in the accuracy of methods of prediction and the ever increasing speed of computers, time based predictions can be readily provided for any major item of infrastructure which is to be mined beneath.

There are a number of methods of predicting subsidence, including Empirical Methods, Profile Function Methods, Influence Function Methods, Numerical Modelling Methods, and Graphical Methods. To provide time based predictions, the method of prediction must be capable of determining the predicted movements at any point within the mining area, rather than just determining the maximum movements over the mining area.

One method which can be used to make time based predictions is the Incremental Profile Method, which was the method used for the examples in this paper. The Incremental Profile Method was developed by Mine Subsidence Engineering Consultants (MSEC) in the latter part of 1994, and has been continuously improved over time.

The method initially used a number of prediction lines, orientated perpendicular to the longwall panels, to make predictions of subsidence, tilt and strain across the longwalls. The predictions along each line were made based on a library of standard profiles obtained from observations at a number of collieries in the Southern, Newcastle, Hunter and Western Coalfields of New South Wales.

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At the time of this report, the library consisted of 693 different half-profile shapes for single-seam mining situations, and 236 different half-profile shapes for multi-seam mining situations. The shapes of the observed subsidence profiles vary for areas with differing geologies and, therefore, site specific predictions are undertaken where local monitoring data is available.

The prediction lines provide profiles of subsidence, tilt and strain across the longwalls, based on the local depth of cover, extracted seam thickness, geology, and longwall dimensions. The predicted subsidence, tilt and strain contours across the mining area are then determined from the profiles along the prediction lines, using a proprietary contouring program such as Surfer.

In the last two years, the Incremental Profile Method has been refined so that predictions can be made at specific points across the mining area, rather than along prediction lines, which allowed some automation of the prediction process. The method predicts subsidence, tilt and strain at any point within the mining area, based on the local depth of cover, extracted seam thickness, geology and longwall dimensions, using the same library of profile shapes.

The Incremental Profile Method can be used to make predictions on a grid of points across the mining area, which can then be used to make predicted subsidence, tilt and strain contours over the longwalls. The Incremental Profile Method can also be used to directly make predictions along the alignments of items of infrastructure.

The Incremental Profile Method has recently been transferred into C++ programming language which has dramatically increased the speed of calculation. This has allowed the prediction of subsidence contours across the mining area, and hence predictions at major items of infrastructure, to be readily determined for varying longwall extraction face positions.

Time based predictions require that subsidence contours are determined for set increments of longwall travel. The chosen increment of the extraction face is dependant on a number of factors, including maximum predicted subsidence, the sensitivity of major infrastructure to subsidence movements, and the proposed rate of mining. An increment of between 20 and 50 metres has typically been adopted in the past.

The shape of the predicted subsidence contours above the extraction face is dependant on a number of factors including the geology, mining geometry and the rate of extraction. At very slow rates of extraction, the shape of the subsidence travelling wave above the extraction face is similar to the shape above the finishing end of a longwall panel which has a similar geology and mining geometry. At faster rates of extraction, the shape of the subsidence travelling wave above the extraction face is flatter, and the resulting longitudinal travelling tilts and strains are less.

There is limited amount of observed data for longwall travelling waves for varying geologies, mining geometries and extraction rates. A conservative approach is to adopt the finishing end subsidence profile for the travelling wave at the extraction face, which provides upperbound predictions for the travelling tilts and strains. A more accurate representation would be to adopt a subsidence profile at the extraction face which has a slope of between 50 % and 90 % of the finishing end of subsidence profile, depending on the rate of extraction.

The predicted subsidence movements at the major items of infrastructure can be determined from the predicted subsidence contours over the mining area. It is possible, using the Incremental Profile Method, to make predictions directly at each item of infrastructure, rather than determining the predictions from the subsidence contours over the mining area. However, the subsidence contours over the mining area can be more easily reviewed and verified than if predictions are made directly at the major items of infrastructure.

EXAMPLES OF TIME BASED PREDICTIONS

An example of time based predictions has been made for a generic longwall layout consisting of three longwalls. The predicted subsidence contours and the predicted profiles of subsidence along the longitudinal axes of the longwalls, at four increments of the second longwall extraction face position, are provided in Figure 1. The predicted profiles of subsidence along the transverse line, at 50 metre increments of the extraction face for each longwall, are provided in Figure 2.
Fig. 1 - Predicted Subsidence Contours and Predicted Subsidence Profiles along the Longitudinal Axes of the Longwalls during the Extraction of the Second Longwall.
(The rectangles indicate the outline of the longwall extraction area. The thick lines that are oriented longitudinal to the longwalls indicate the location of the prediction lines for this figure. The graphs show the predicted subsidence profiles along these prediction lines.)
The predicted subsidence contours and the predicted profiles of subsidence along the transverse and longitudinal lines show the progressive development of subsidence during extraction of the longwalls.

**APPLICATION OF TIME BASED PREDICTIONS**

Time based predictions have been made at a number of major items of infrastructure in the past. Two examples are provided in this paper: the Main Southern Railway at Tahmoor Colliery and the gas and water pipelines across an unnamed Creek at West Cliff Colliery.

An early use of time based predictions was made in 1998 for the Cataract Tunnel at Appin Colliery. It was originally intended that predictions were to be made for 50 metre increments of the longwall extraction face; however, this was reduced to a total of six increments due to the amount of calculation involved. With current methods of prediction, however, the 50 metre increments can be calculated in less time than the six increments took in 1998.
The Main Southern Railway is located adjacent to Longwall 23A at Tahmoor Colliery. The location of the railway and the longwalls at Tahmoor Colliery are shown in Figure 3. The Incremental Profile Method was used to determine the predicted incremental subsidence contours, due to the extraction of Longwall 23A, at 50 metre increments of the extraction face position. The actual subsidence along the railway was monitored during the extraction of this longwall, and a comparison between the maximum observed subsidence and maximum predicted subsidence along the railway is provided in Figure 4.

Fig. 3 - Location of Longwall 23A and the Main Southern Railway at Tahmoor Colliery

Fig. 4 - Comparisons between maximum predicted and maximum observed subsidence movements along the Main Southern Railway due to Tahmoor Longwall 23A
It can be seen from the previous figure that the observed movements were generally less than those predicted during the extraction of Longwall 23A. The observed movements only exceeded the predicted movements at one small moment in time by less than 3 mm, which is extremely small, and the maximum observed subsidence was less than the maximum predicted subsidence at the completion of mining.

Three natural gas pipelines and one water pipeline were mined beneath by Longwall 30 at West Cliff Colliery. The pipelines were subjected to both systematic subsidence movements, and to valley related upsidence and closure movements, where the pipelines cross Unnamed Creek. The locations of the Longwall 30, the pipeline easement and Unnamed Creek are shown in Figure 5.

![Fig. 5 - Locations of Longwall 30, the pipeline easement and unnamed creek at West Cliff Colliery](image)

The maximum predicted subsidence, upsidence and closure along the pipeline easement were determined for increments of Longwall 30 extraction face position. A comparison between the maximum predicted and maximum observed subsidence, upsidence and closure movements along the easement, during the extraction of Longwall 30, are provided in Figure 6, Figure 7 and Figure 8, respectively.

The observed subsidence, upsidence and closure movements were generally less than the predicted movements during the extraction of Longwall 30. The observed subsidence and closure movements were initially slightly greater than predicted, however, the movements at this stage of mining were very small and naturally more difficult to predict.

It can be seen from the examples in this paper that time based predictions can provide a useful tool as part of the overall management strategy where major items of infrastructure are mined beneath. Time based predictions can be readily provided for any major item of infrastructure using current methods of subsidence prediction.
Fig. 6 - Comparisons between maximum predicted and maximum observed subsidence Movements along the pipeline easement above Longwall 30 at West Cliff Colliery

Fig. 7 - Comparisons between maximum predicted and maximum observed upsidence Movements along the pipeline easement above Longwall 30 at West Cliff Colliery

Fig. 8 - Comparisons between Maximum predicted and maximum observed closure movements along the pipeline easement above Longwall 30 at West Cliff Colliery
ACKNOWLEDGEMENTS

The authors of this paper would like to thank Illawarra Coal and Centennial Coal Tahmoor for providing information relating to mining under pipelines and railways.

REFERENCES

NUMERICAL MODELLING OF MINING INDUCED SUBSIDENCE

Walter Keilich¹, Ross Seedsman², and Naj Aziz¹

ABSTRACT: A methodology of subsidence prediction using the Distinct Element code UDEC has been developed as an alternative for subsidence modelling in the Southern Coalfield, New South Wales, Australia. The models have been validated by comparison with empirical results and observed caving behaviour. At this stage, modelling capability is limited to flat lying terrain. It is planned to apply the methodology to areas of high topographical relief to investigate the mechanics of valley closure.

INTRODUCTION

Ground subsidence due to mining has been the subject of intensive research for several decades, and it remains to be an important topic confronting the mining industry today. In the Southern Coalfield of NSW, Australia, there is particular concern about subsidence impacts on incised river valleys – valley closure, upsidence, and the resulting localised loss of surface water under low flow conditions. Most of the reported cases have occurred when the river valley is directly undermined. However, there are a number of cases where closure and upsidence is reported above unmined coal. These latter events are especially significant as they influence decisions regarding stand-off distances and hence mine layouts and reserve recovery.

The deformations of the valleys indicate the onset of locally compressive stress conditions. Compressive conditions are anticipated when the surface deforms in a sagging mode, for example directly above the longwall extraction: they are not expected when the surface deforms in a hogging mode. To date, explanations for valley closure under the hogging mode have considered undefined compressive stress redistributions in the horizontal plane, or block translations from the sagging mode. This research is investigating the possibilities of the block translation model.

Subsidence prediction in Australia is currently limited to empirical and numerical techniques. The empirical techniques are suitable for flat lying or gently sloping areas but are unsuitable for areas of large topographical relief. From the available numerical techniques, FLAC has been commonly used for assessing the impacts of longwall mining on river valleys. FLAC has limited application as the code is not capable of modelling discontinuous rock masses effectively.

In this project, a methodology of subsidence prediction using the Distinct Element code UDEC is being developed as an alternative for subsidence modelling in the Southern Coalfield. The UDEC models have been validated by comparison with empirical results and comparison of observed caving behaviour. The expected outcomes will include a reliable subsidence prediction tool capable of simulating ground deformations and sub surface movements in flat terrain and river valleys, and a more complete understanding of valley closure. This paper will present work completed to date.

SUBSIDENCE IN THE SOUTHERN COALFIELD

During longwall mining, a large void in the coal seam is produced and this disturbs the equilibrium conditions of the surrounding rock strata, which bends downward while the floor heaves. When the goaf reaches a sufficient size, the roof strata will fail and cave. Seedsman (2004) reports that caving does not necessarily occur vertically above the extracted coal panel, but in many cases, caving is defined by a goaf angle that trends over the goaf. This angle is most likely a function of the bedding structure of the roof and the orientation of the goaf with respect to sub vertical jointing.

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² Visiting Fellow, University Of Wollongong
In the Newcastle Coalfield the average goaf angle is 12º with a standard deviation of 8º. Numerical modelling by the CSIRO (1999) of the caving in the Southern Coalfield appears to support a goaf angle value of 12º. Further numerical modelling by Gale (2005) in an unspecified coalfield also supports this value. Caving will cease when the goaf angle encounters a stratigraphic unit strong enough to bridge what is now the effective span. This concept is illustrated in Figure 1. The goaf and overburden strata will then compact over time and become stabilised.

![Fig. 1 - Relationship between panel width, goaf angle and effective span](image1)

The caving of the roof strata as previously described, gives rise to several zones within the overburden strata. The number of zones varies in the literature with Kratzsch (1983) describing six zones, Peng (1992) describing four zones, and Kapp (1984) describing three zones. These zones are not distinct but there is a gradual transition from one to another. Seedsman (2004) reported on the existence of a massive unit in the strata of the Newcastle Coalfield and presented an alternative way of predicting subsidence based on the Voussoir Beam analogue. For this method to be applied, it is assumed that the massive unit remains elastic and all goafing takes place underneath the massive unit. Therefore the developed subsidence is a function of the deflection of the massive unit, provided the massive unit remains elastic and does not fail. Unfortunately, the amount of information on the caving characteristics in the Southern Coalfield is somewhat limited. Microseismic results from an Australian Coal Association Research Program (ACARP) project (CSIRO, 1999) provided some useful information on the caving behaviour at Appin Colliery, which is located in the Southern Coalfield. The longwall panel that was monitored was 200 m wide and extracted the 2.3 m thick Bulli Seam at a depth of about 500 m. The monitoring included the installation of 17 triaxial geophones and nine geophones in a borehole drilled from the surface to the Bulli Seam and two perpendicular surface strings of four geophones each. The period of monitoring was approximately four months, during which there was 700 m of face retreat. From the monitoring it was seen that the majority of fracturing extended approximately 50 m to 70 m above the Bulli Seam with no fracturing exceeding approximately 290 m, and to a depth of 80 m to 90 m into the floor. Figure 2 illustrates the microseismic events in a cross section of the monitored longwall.

![Fig. 2 - Cross section of longwall with microseismic event location (CSIRO, 1999)](image2)
An analysis of the stratigraphic details in the subsidence handbook by Holla and Barclay (2000) shows that the Bulgo Sandstone is the most massive unit in the stratigraphy of the Southern Coalfield, with a thickness ranging from approximately 90 m to 200 m, and located at a distance between 90 m and 120 m above the Bulli Seam at Appin Colliery. It is also the strongest of the larger upper units as indicated by a geotechnical characterization (MacGregor and Conquest, 2005). If the position of the Bulgo Sandstone were overlain onto Figure 2, it would be seen that the majority of the fracturing in the goaf is contained by the Bulgo Sandstone. This would seem to suggest that the Bulgo Sandstone is acting as the massive spanning unit, therefore all potential subsidence development can be theoretically derived from a voussoir analysis of the Bulgo Sandstone.

VALLEY CLOSURE

ACARP (2002) contains a comprehensive literature review on valley bulging, along with an empirical method to predict valley closure, upsidence, compressive strain and regional horizontal movement for river valleys that have been undermined. It is proposed that during the formation of a river valley, the horizontal stresses in the valley sides redistribute to the valley base, causing an increase in horizontal stress. Bulging of the valley base is a result of this stress redistribution and is a natural phenomenon. When a river valley is undermined, the horizontal stresses are redistributed from the cave zone to the surface. This results in a further increase of the horizontal stress in the valley base. If the elevated horizontal stress exceeds rock strength, the valley base will fail in compression and buckle up-wards or over-ride adjacent stratum. Failure of the valley base continues downward until equilibrium is achieved. This failure of the strata in the base of the valley allows some relaxation of the sides of the valley to occur, causing closure of the valley sides.

For a river valley that is directly undermined by a longwall, the above-mentioned explanation is valid. Results from the empirical study (ACARP, 2002) show that valley closure occurs well outside the goaf edge, up to a longitudinal distance of 1500 m from the end of the longwall. It would be expected that a valley in the convex part of the subsidence profile would open up, not close as seen by the empirical results. Whether this valley closure is driven by the magnitude of horizontal stress or the magnitude of the tilt in the subsidence profile is uncertain and is anticipated to be clarified by numerical modelling.

EMPIRICAL PREDICTIONS

The method devised by the New South Wales Department of Primary Industries has been in existence since 1985 and is available as a handbook (Holla and Barclay, 2000). Since then, the method has been refined with the addition of subsidence data, and a discussion on the effects of mining induced subsidence on public utilities, dwellings and water bodies. Whilst not accounted for in the prediction technique, there is also a discussion on the major factors modifying the theoretical subsidence behaviour such as faults, dykes, and gullies. Several case studies were also presented to illustrate these factors in action.

The subsidence data and resulting graphs in this method were obtained from collieries in the area between the Illawarra Escarpment and the Burragorang Valley. This data was collected over a period of thirty years. The majority of the mines included in the analyses were mining the Bulli seam except in two cases for which the workings were in the Wongawilli seam. The predominant method of mining was by longwall mining, although some pillar extraction data has been included. The relationship between $S_{max}/T$ and $W/H$ for single panels is illustrated in Figure 3.

It can be seen from Figure 3 that the lower curve represents the relationship between the width to cover depth ($W/H$) and subsidence factor ($S_{max}/T$) for longwall extraction, where $S_{max}$ is the maximum developed subsidence and $T$ is the extracted thickness. It can also be seen from Figure 3 that the largest longwall $W/H$ ratio still falls into the sub-critical category ($W/H < 1.4$). This is a result of the deep mining conditions in the Southern Coalfield, and although data exists for $W/H$ ratios between 0.5 and 0.9, the resulting scatter suggests that subsidence prediction would be more accurate for $W/H$ ratios less than 0.5.
NUMERICAL MODELLING STRATEGY

The approach used in the numerical modelling was to try and replicate the trends in Figure 3 before extending the numerical modelling to undermined river valleys in an effort to understand the mechanisms behind valley closure. Holla and Barclay (2000) contain a list of mines and extraction details, from which the ground movement data were collected and the subsidence curves derived (single panel only). The majority of the mines extracted the Bulli Seam using the longwall method of mining. The data that was derived from pillar extraction and Wongawilli Seam extraction was excluded from the modelling. It should be noted that the extraction details are approximate figures only.

Holla and Barclay (2000) also contain the thickness of the stratigraphic units in the overburden, grouped according to colliery. This was used for the derivation of the thickness of rock units above the Bulli seam for different mines. Excluding mines that utilise pillar extraction, extract the Wongawilli Seam, it was concluded that a minimum of three models can be created from the available data (Table 1).

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Individual Panel Width W (m)</th>
<th>Cover Depth H (m)</th>
<th>Extracted Thickness (m)</th>
<th>W/H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>105</td>
<td>413</td>
<td>2.7</td>
<td>0.25</td>
</tr>
<tr>
<td>Model 2</td>
<td>158</td>
<td>450</td>
<td>2.5</td>
<td>0.35</td>
</tr>
<tr>
<td>Model 3</td>
<td>160</td>
<td>288</td>
<td>3.0</td>
<td>0.56</td>
</tr>
</tbody>
</table>

It must be noted that although 18 potential models can be created with the available data, three models was considered sufficient to cover the range of W/H ratios represented in the single panel subsidence curve in Figure 3. At the time of writing, another model with a W/H ratio of 0.81 was running but early indications suggest a model this large is impractical to run, with the current run time of this model exceeding two weeks.

Model Geometry

Symmetry has been utilised to halve the size of the models needed, with the right hand side of the model representing the centreline of the panel. Each model has the left hand boundary fixed at five times the excavation width, as indicated by the UDEC user’s manual (Itasca, 2000), or the predicted range of ground movement as indicated by the 29° angle of draw (Holla and Barclay, 2000), or whichever is the greater value. The stratigraphic thickness for each rock unit in the Southern Coalfield is given in Table 2 and the finalised dimensions for each model are given in Table 3. Bedding planes were assumed as horizontal and vertical joints were placed with a 90° dip and offset to form a brickwork style pattern.
Table 2 - Thickness of stratigraphic units for each model, in descending order

<table>
<thead>
<tr>
<th>Stratigraphic Unit</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>88 153 78</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>20 13 7</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>34 23 12</td>
</tr>
<tr>
<td>Bulgo Sandstone</td>
<td>145 156 92</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>40 23 11</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>50 32 36</td>
</tr>
<tr>
<td>Wombarra Shale</td>
<td>16 29 29</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>20 21 23</td>
</tr>
<tr>
<td>Bulli Seam</td>
<td>2.7 2.5 3</td>
</tr>
<tr>
<td>Loddon Sandstone</td>
<td>8 8 8</td>
</tr>
<tr>
<td>Balgownie Seam</td>
<td>1 1 1</td>
</tr>
<tr>
<td>Lawrence Sandstone</td>
<td>4 4 4</td>
</tr>
<tr>
<td>Cape Horn Seam</td>
<td>2 2 2</td>
</tr>
<tr>
<td>UN2*</td>
<td>6 6 6</td>
</tr>
<tr>
<td>Hargraves Coal Member</td>
<td>0.1 0.1 0.1</td>
</tr>
<tr>
<td>UN3*</td>
<td>10 10 10</td>
</tr>
<tr>
<td>Wongawilli Seam</td>
<td>10 10 10</td>
</tr>
<tr>
<td>Kembla Sandstone</td>
<td>3 3 3</td>
</tr>
<tr>
<td>Lower Coal Measures</td>
<td>50 50 50</td>
</tr>
<tr>
<td>Total Depth</td>
<td>509.8 546.6 385.1</td>
</tr>
</tbody>
</table>

*UN-NAMED MEMBER

Table 3 - Finalised width and depth for each model

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Total Model Width (m)</th>
<th>Total Model Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>315</td>
<td>509.8</td>
</tr>
<tr>
<td>Model 2</td>
<td>474</td>
<td>546.6</td>
</tr>
<tr>
<td>Model 3</td>
<td>480</td>
<td>385.1</td>
</tr>
</tbody>
</table>

Material Properties

A great deal of information has been published on the material properties of the stratigraphic units above and including the Bulgo Sandstone by Pells (1993). Most of this data is derived from civil engineering works in and around Sydney, not specifically the Southern Coalfield. Most recently, a drilling program has been completed which contains the geotechnical characterisation of several boreholes that were drilled over Appin and Westcliff collieries (MacGregor and Conquest, 2005). As a result of this geotechnical characterisation and a survey of the literature (CSIRO, 2002; Williams and Gray, 1980; and McNally, 1996) a complete set of material properties have been derived (Table 4). The material properties that have been derived from laboratory testing have been used directly in the models without calibration or modification.

Table 4 - Selected material properties for stratigraphic units

<table>
<thead>
<tr>
<th>Stratum</th>
<th>E (GPa)</th>
<th>ν</th>
<th>c  (MPa)</th>
<th>φ  (°)</th>
<th>σ_T (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>13.99</td>
<td>0.29</td>
<td>9.70</td>
<td>37.25</td>
<td>3.58</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>11.65</td>
<td>0.25</td>
<td>8.85</td>
<td>35.00</td>
<td>3.40</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>10.37</td>
<td>0.46</td>
<td>10.60</td>
<td>27.80</td>
<td>2.90</td>
</tr>
<tr>
<td>Bulgo Sandstone</td>
<td>18.00</td>
<td>0.23</td>
<td>17.72</td>
<td>35.40</td>
<td>6.55</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>19.20</td>
<td>0.26</td>
<td>14.57</td>
<td>27.80</td>
<td>4.83</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>20.57</td>
<td>0.23</td>
<td>13.25</td>
<td>40.35</td>
<td>7.18</td>
</tr>
<tr>
<td>Wombarra Shale</td>
<td>17.00</td>
<td>0.37</td>
<td>14.51</td>
<td>27.80</td>
<td>4.81</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>23.78</td>
<td>0.22</td>
<td>19.40</td>
<td>33.30</td>
<td>7.87</td>
</tr>
<tr>
<td>Bulli Seam</td>
<td>2.80</td>
<td>0.30</td>
<td>6.37</td>
<td>25.00</td>
<td>0.84</td>
</tr>
</tbody>
</table>
Table 4 - Selected material properties for stratigraphic units (continued)

<table>
<thead>
<tr>
<th></th>
<th>E (GPa)</th>
<th>υ</th>
<th>c (MPa)</th>
<th>φ (°)</th>
<th>σT (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loddon Sandstone</td>
<td>15.07</td>
<td>0.33</td>
<td>17.10</td>
<td>28.90</td>
<td>5.65</td>
</tr>
<tr>
<td>Balgownie Seam</td>
<td>2.80</td>
<td>0.30</td>
<td>6.37</td>
<td>25.00</td>
<td>0.84</td>
</tr>
<tr>
<td>Lawrence Sandstone</td>
<td>15.07</td>
<td>0.33</td>
<td>17.10</td>
<td>28.90</td>
<td>5.65</td>
</tr>
<tr>
<td>Cape Horn Seam</td>
<td>2.00</td>
<td>0.30</td>
<td>2.87</td>
<td>25.00</td>
<td>0.70</td>
</tr>
<tr>
<td>UN2</td>
<td>13.48</td>
<td>0.25</td>
<td>19.89</td>
<td>28.90</td>
<td>6.74</td>
</tr>
<tr>
<td>Hargraves Coal Member</td>
<td>2.80</td>
<td>0.30</td>
<td>6.37</td>
<td>25.00</td>
<td>0.84</td>
</tr>
<tr>
<td>UN3</td>
<td>13.00</td>
<td>0.25</td>
<td>19.18</td>
<td>28.90</td>
<td>6.50</td>
</tr>
<tr>
<td>Wongawilli Seam</td>
<td>2.00</td>
<td>0.30</td>
<td>2.87</td>
<td>25.00</td>
<td>0.70</td>
</tr>
<tr>
<td>Kembla Sandstone</td>
<td>18.15</td>
<td>0.28</td>
<td>18.02</td>
<td>28.90</td>
<td>6.11</td>
</tr>
<tr>
<td>Lower Coal Measures</td>
<td>9.37</td>
<td>0.29</td>
<td>12.20</td>
<td>27.17</td>
<td>3.75</td>
</tr>
</tbody>
</table>

Where,

E = Young’s Modulus
υ = Poisson’s Ratio
c = Cohesion
φ = Friction Angle
σT = Tensile Strength

Bedding Planes and Properties

Bedding, stratification or layering is one of the most fundamental and diagnostic features of sedimentary rocks. In numerical modelling, it is important to correctly distinguish what constitutes bedding planes and intrabed structures as bedding planes are the major source of shear and slip in a discontinuous rock mass.

Bedding is due to vertical differences in lithology, grain size, grain shape, packing or orientation. Generally, bedding is layering within beds on a scale of about 1 or 2 cm, and lamination is layering within beds on a scale of 1 or 2 mm (Tucker, 2003; and Selley, 2000). Limited information exists about bedding planes in the Southern Coalfield. Most of the information has been derived from civil engineering works and visual examination of outcrops along the coast by Ghobadi (1994). It is also recognised that strata thickness and bedding plane thickness will vary from site to site, so it would be advantageous to derive the required information from a complete geotechnical investigation at one site, if possible.

The drill cores that were obtained for the geotechnical characterisation (MacGregor and Conquest, 2005) were logged for discontinuities, but unfortunately bedding planes or drilling induced fractures were not specifically identified. The authors were allowed access to the logs and laboratory reports. Neutron and gamma logging was also performed on holes. A site visit was conducted by the authors and a visual examination of the core, along with a comparison of the logs was carried out for the Bulgo Sandstone. It was found that there was a good correlation between major bedding planes and partings identified in the core and the corresponding logs. When compared to data provided by Pells (1993) and Ghobadi (1994), there was good agreement apart from the Newport Formation and Bald Hill Claystone. In these instances, it was decided to use the values provided by Pells (1993). The bedding plane spacings used in the models are summarised in Table 5.

Table 5 - Bedding plane spacing

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>Bedding Plane Spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>9</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>1</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>0.3</td>
</tr>
<tr>
<td>Bulgo Sandstone</td>
<td>9</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>3</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>4</td>
</tr>
<tr>
<td>Wombarra Claystone</td>
<td>3</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>3</td>
</tr>
</tbody>
</table>
Information on specific bedding plane properties are scarce and if the discontinuities are not directly laboratory tested, estimates or values from field studies have to be used. Derivation of the joint and normal and shear stiffness was done in accordance to the procedures described by Itasca (2000). It seems that the shear stiffness can be approximated as one-tenth of the normal stiffness. This approach has been used by Itasca (2000), and has been used by Coulthard (1995) and Badelow et al (2005). The derived joint normal and shear stiffness used for each rock unit is shown in Table 6.

The joint and bedding plane strength parameters have been derived from Chan, Kotze and Stone (2005), and Barton (1976) has been used to calculate cohesion based on the JRC and JCS values given by Chan, Kotze and Stone (2005). The bedding plane properties used in the models can be seen in Table 7.

Vertical Joints and Properties

Very little data exists on the vertical joint spacing in rock units in the Southern Coalfield, and even where geotechnical characterisations have been completed; vertical joint spacing simply cannot be assessed from HQ cores.

Price (1966) reports on work done in Wyoming, USA, which suggests for a given lithological type, the concentration of joints is inversely related to the thickness of the bed. Examples were given for dolomite where joints in a 10 ft. thick bed occurred at every 10 ft.; and joints in a 1 ft. thick bed occurred every 1 ft. Similar results were also reported for sandstone and limestone. The mechanism proposed by Price (1966) assumed that the cohesion between adjacent beds is non-existent and that friction angle; normal stress and tensile strength are all constant. It was suggested that while these parameters will change in reality, these factors cause only second-order variations in the relationship between joint frequency and bed thickness. A comprehensive review of the Price model was performed by Mandl (2005). In addition, this review also included Hobbs’ model, which is a more complex model that takes into account the elastic modulus and bedding plane cohesion of adjacent beds. Both models predict a joint spacing that scales with bed thickness.

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>Normal Stiffness (GPa/m)</th>
<th>Shear Stiffness (GPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hawkesbury Sandstone</td>
<td>21</td>
<td>2.1</td>
</tr>
<tr>
<td>Newport Formation</td>
<td>140</td>
<td>14</td>
</tr>
<tr>
<td>Bald Hill Claystone</td>
<td>204</td>
<td>20.4</td>
</tr>
<tr>
<td>Bulgo Sandstone</td>
<td>26</td>
<td>2.6</td>
</tr>
<tr>
<td>Stanwell Park Claystone</td>
<td>78</td>
<td>7.8</td>
</tr>
<tr>
<td>Scarborough Sandstone</td>
<td>76</td>
<td>7.6</td>
</tr>
<tr>
<td>Wombarra Claystone</td>
<td>115</td>
<td>11.5</td>
</tr>
<tr>
<td>Coal Cliff Sandstone</td>
<td>108</td>
<td>10.8</td>
</tr>
</tbody>
</table>

Ghobadi (1994) reports that the vertical joint spacing in the Hawkesbury Sandstone is observed to be 2-5 m, the Scarborough Sandstone 1-4 m, the Bulgo Sandstone 0.5-1.5 m, the Stanwell Park Claystone 0.1-0.5 m, and the Wombarra Claystone 0.2-0.6 m apart. It was noted that many of the joints on the escarpment and coastline are filled with calcite and/or clay. These values are not in good agreement with the Price joint model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Bedding Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle (°)</td>
<td>28</td>
</tr>
<tr>
<td>Residual Friction Angle (°)</td>
<td>15</td>
</tr>
<tr>
<td>JCS</td>
<td>4</td>
</tr>
<tr>
<td>JRC</td>
<td>5</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>0.7</td>
</tr>
<tr>
<td>Residual Cohesion (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>0</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>0</td>
</tr>
</tbody>
</table>
Pells (1993) reports that the vertical joint spacing in the Hawkesbury Sandstone is 7-15 m in the Southern catchment area, the Newport Formation 1-3 m, Bald Hill Claystone 1 m, and the Bulgo Sandstone 2-13 m. These values are in good agreement with the Price joint model, therefore it was assumed that vertical joint spacing is equal to bed thickness and this assumption was used in the numerical models. Vertical joint properties have been estimated in the same manner as for bedding planes. The vertical joint properties are shown in Table 8.

<table>
<thead>
<tr>
<th>Property</th>
<th>Vertical Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle (°)</td>
<td>28</td>
</tr>
<tr>
<td>Residual Friction Angle (°)</td>
<td>15</td>
</tr>
<tr>
<td>JCS</td>
<td>2</td>
</tr>
<tr>
<td>JRC</td>
<td>8</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>1</td>
</tr>
<tr>
<td>Residual Cohesion (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>0</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>0</td>
</tr>
</tbody>
</table>

In-Situ Stress
A thorough review of regional and local in-situ stress has been compiled by the CSIRO (2002) for their numerical modelling. From 206 measurements across the Sydney Basin, the ratio of horizontal stress to vertical stress was found to be in the range of 1.5-2.0. For the numerical models, a horizontal to vertical stress ratio of two was implemented.

Mesh Generation
The mesh employed was relatively simple. Each block was subdivided into four constant strain zones. It was noted by Coulthard (1995) that this may result in a unit of large blocks excessively stiffer than a unit of smaller blocks. This is particularly noticeable where the larger unit overlies the smaller one. If this occurs in the models, the mesh density will be increased in the areas of interest.

Constitutive Models
The constitutive model employed is the Mohr-Coulomb model. The constitutive model used for the joints is the Mohr-Coulomb residual strength model. This joint model has the capability to reduce or increase friction, cohesion, dilation and tensile strength.

RESULTS
Three models (Models 1, 2 and 3) had been run and analysed. A fourth model representing a W/H ratio of 0.81 was running at the time of writing but its excessive run times may rule it out in any further analysis. The results have been analysed and plots produced for:

- $S_{\text{max}}/T$ (subsidence factor),
- $S_{\text{goaf}}/S_{\text{max}}$,
- $K_1$ (maximum tensile strain constant),
- $K_2$ (maximum compressive strain constant),
- $K_3$ (maximum tilt constant), and
- $D/H$ (position of inflection point relative to goaf).

Strain and tilt are defined by the equation (Holla and Barclay, 2000):

$$+ E_{\text{max}} - E_{\text{max}} = 1000 \times K \times \frac{S_{\text{max}}}{H}$$
Where,

\[ + E_{\text{max}} = \text{Max tensile strain} \]
\[ - E_{\text{max}} = \text{Max compressive strain} \]
\[ G_{\text{max}} = \text{Max tilt} \]
\[ K = \text{Constant} \]
\[ H = \text{Depth of cover} \]

Horizontal strain is the change in length per unit of the original horizontal length of ground surface. Tensile strains occur in the trough margin and over the goaf edges. Compressive strains occur above the extracted area. Holla and Barclay (2000) noted that maximum tensile strains are generally not larger than 1 mm/m and maximum compressive strains 3 mm/m, excluding topographical extremes.

Tilt of the ground surface between two points is found by dividing the difference in subsidence at the two points by the distance between them. Maximum tilt occurs at the point of inflection where the subsidence is roughly equal to one half of \( S_{\text{max}} \).

The point of inflection is the location where tensile strains become positive and vice versa. It has been found by Holla and Barclay (2000) that the inflection point lies inside the goaf for W/H ratios greater than 0.5.

The respective maximum values were readily picked from the model outputs. The strain profiles for Models 2 and 3 contained anomalies where strain turned compressive in two sections of the profile above unmined coal. However, the magnitude of the strains was extremely low and this behaviour has been ascribed to the modelling technique.

Block failure and the formation of the caved zone can be seen in Figure 4. Block failure trends inward over the goaf at an angle of approximately 12° to 15°. This is in good agreement with the CSIRO (1999) and Gale (2005). The caved zone also stops abruptly at the base of the Bulgo Sandstone; this is in general agreement with the microseismic monitoring (CSIRO, 1999).

Slip occurs on every bedding plane up to the surface, and vertical joints open up in the caved zone and also along the surface, outside the goaf edge.

![Fig. 4 - Typical cave zone above longwall panel](image)

The analysed results from Models 1, 2 and 3 are shown below in Table 9.
Table 9 - Results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model 1</th>
<th>Model 2</th>
<th>Model 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>W (m)</td>
<td>105</td>
<td>158</td>
<td>160</td>
</tr>
<tr>
<td>H (m)</td>
<td>413</td>
<td>450</td>
<td>288</td>
</tr>
<tr>
<td>T (m)</td>
<td>2.7</td>
<td>2.5</td>
<td>3.0</td>
</tr>
<tr>
<td>W/H</td>
<td>0.25</td>
<td>0.35</td>
<td>0.56</td>
</tr>
<tr>
<td>$S_{\text{max}}$ (mm)</td>
<td>41.12</td>
<td>162.39</td>
<td>312.72</td>
</tr>
<tr>
<td>$S_{\text{goaf}}$ (mm)</td>
<td>39.64</td>
<td>82.64</td>
<td>87.24</td>
</tr>
<tr>
<td>$+E_{\text{max}}$ (mm/m)</td>
<td>0.092</td>
<td>0.139</td>
<td>0.690</td>
</tr>
<tr>
<td>$-E_{\text{max}}$ (mm/m)</td>
<td>0.065</td>
<td>0.287</td>
<td>0.516</td>
</tr>
<tr>
<td>$G_{\text{max}}$ (mm/m)</td>
<td>0.086</td>
<td>1.275</td>
<td>3.731</td>
</tr>
<tr>
<td>D (m)</td>
<td>-96.00</td>
<td>5.50</td>
<td>18.50</td>
</tr>
<tr>
<td>$S_{\text{max}}/T$</td>
<td>0.015</td>
<td>0.065</td>
<td>0.104</td>
</tr>
<tr>
<td>$S_{\text{goaf}}/S_{\text{max}}$</td>
<td>0.964</td>
<td>0.509</td>
<td>0.279</td>
</tr>
<tr>
<td>K1</td>
<td>0.924</td>
<td>0.386</td>
<td>0.635</td>
</tr>
<tr>
<td>K2</td>
<td>0.653</td>
<td>0.794</td>
<td>0.475</td>
</tr>
<tr>
<td>K3</td>
<td>0.864</td>
<td>3.533</td>
<td>3.436</td>
</tr>
<tr>
<td>D/H</td>
<td>-0.232</td>
<td>0.012</td>
<td>0.064</td>
</tr>
</tbody>
</table>

To put the results into perspective, the results from Table 9 are reproduced on the corresponding empirical curves from Holla and Barclay (2000). These are shown below in Figures 5, 6, 7, 8, 9 and 10.

Fig. 5 - Model results for $S_{\text{max}}/T$ (after Holla and Barclay, 2000)

It can be seen from Figures 5 and 6 that the numerical models predict maximum developed subsidence and goaf edge subsidence quite well. Given the amount of scatter in the empirical data for the subsidence values, this is a good result.
Fig. 6 - Model results for $S_{goaf}/S_{max}$ (after Holla and Barclay, 2000)

Fig. 7 - Model results for K1 (after Holla and Barclay, 2000)

Fig. 8 - Model results for K2 (after Holla and Barclay, 2000)
Strain has been recognised as one of the most difficult parameters to predict due to vertical joints potentially opening up on the surface and the large effect that variations in topography has on the strain profile. Observed strain profiles in the field are never as perfect as theoretical strain profiles due to these factors.

It can be seen from Figures 7 and 8, the model results contain considerable scatter in the data points, as do the empirical results for the strain constants. Part of the problem is the use of the K1 and K2 constant which normalise strains to depth and $S_{max}$ – this may not be valid for subcritical extraction.

The model results for tilt and its associated constant produced good matches with the empirical results. The model results for the tilt constant can be seen in Figure 9.

The results of the position of the infection point relative to the goaf can be seen in Figure 10. It is noted by Holla and Barclay (2000) that the position of the infection point falls inside the goaf for W/H ratios greater than 0.5 or outside the goaf for W/H ratios less than 0.5. It can be seen that this observation holds true for Model 1 (W/H = 0.25) and Model 3 (W/H = 0.56). The location of the inflection point is within 32 m of the position of maximum tilt for all three models. The subsidence at the inflection point is roughly one half of $S_{max}$ for all models and this is in agreement with Holla and Barclay (2000).

The calculated angle of draw for the models varies between 19° and 41°. This produced an average value of 30°, which is very close to the average value of 29° stated by Holla and Barclay (2000).
SUMMARY

Due to the ongoing nature of this project, the results presented are preliminary and are encouraging. The main aspects of subsidence development are represented generally well with the numerical modelling. It is anticipated that further verification can be achieved by the application of voussoir beam methods to the Bulgo Sandstone, as it appears to act as a massive elastic unit and the resulting subsidence should be primarily a function of the deflection of this unit.

The next step will be the construction of models that simulate undermined river valleys. These models will be ideally based on the models presented in this paper, and the location of the valley will be varied in its position relative to the centre of the longwall panel. It is anticipated that this modelling will shed some light on the mechanisms behind valley closure.

ACKNOWLEDGMENTS

The authors wish to express their thanks to Seedsman Geotechnics Pty. Ltd. for financial support, BHP Illawarra Coal Pty. Ltd. for access to drill cores, and Strata Control Technology Operations Pty. Ltd. for their cooperation and helpful insights with core logging data and laboratory results. Thanks are also extended to Michael Coulthard of M.A Coulthard and Associates Pty. Ltd. for his technical guidance on the use of UDEC.

REFERENCES


Holla, L and Barclay, E, 2000. Mine subsidence in the Southern Coalfield, NSW, Australia, pp 1-16 (New South Wales Department of Mineral Resources).


IMPACTS OF LONGWALL MINING TO RIVERS AND CLIFFS IN THE SOUTHERN COALFIELD

Daryl Kay\textsuperscript{1}, James Barbato\textsuperscript{1}, Gary Brassington\textsuperscript{2}, and Ben de Somer\textsuperscript{3},

ABSTRACT: Extraction of coal using longwall mining techniques causes subsidence which has potential to affect surface features, including environmentally sensitive areas such as rivers and cliffs.

There are currently a number of proposed extensions to coal mining operations in the Southern Coalfield of New South Wales that are seeking to mine close to rivers and cliffs. These proposals have attracted some community concern at a local and regional level. This concern is largely founded on impacts that have occurred as a result of previous mining activities, the majority of which occurred directly beneath the impacted sites. It was therefore considered a timely exercise to revisit the history of impacts that have occurred as a result of mining close to rivers, particularly where they have occurred in the Southern Coalfield. The rivers reviewed include the Cataract, Nepean, Georges and Bargo Rivers.

Potential effects of longwall mining on clifflines can include rock fracturing; rock falls from cliff lines, riverbed fracturing and water loss. Consideration will be given in this paper to the major mining, geometrical, geotechnical and environmental factors affecting the likelihood of rock falls from cliff lines and riverbed fracturing and water loss, and reference will be made to previous mining experience at collieries in the Southern Coalfield of New South Wales.

Where the mining has not occurred directly beneath rivers, rock fractures, water loss and rock falls from cliff lines have occurred to a much lesser extent when compared to rivers that have been mined directly beneath. The fractures have been observed in local, isolated areas only and were minor in nature. In addition, changes to flow conditions have not been observed in these areas.

A clear understanding of potential impacts from mining of longwalls beneath or near rivers and cliffs is essential for developing relevant baseline studies, assessing potential impacts and formation of appropriate remedial methods. Management plans can then be implemented to monitor and mitigate the identified risks without unduly restricting the extent of mining.

INTRODUCTION

Extraction of coal using longwall mining techniques causes subsidence which has the potential to affect surface features, including environmentally sensitive areas such as rivers and cliffs.

There is an extensive history, in Australia and overseas, of mining directly beneath or close to rivers and cliffs. A number of studies have been conducted that describe many of these past experiences (ACARP, 2002; Holla and Barclay, 2000; Kay D., 1991; and Pells et al, 1987). The majority of these studies have focussed upon mining activities that have occurred directly beneath rivers and cliffs.

There are currently a number of proposed extensions to coal mining operations in the Southern Coalfield of New South Wales that propose to mine close to rivers and cliffs. These proposals have attracted some community concern at a local and regional level. This concern is largely founded on impacts that have occurred as a result of previous mining activities, the majority of which occurred directly beneath the impacted sites. It was therefore considered a timely exercise to revisit the history of impacts that have occurred as a result of mining close to rivers, particularly where they have occurred in the Southern Coalfield. The rivers reviewed include the Cataract, Nepean, Georges and Bargo Rivers.

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\textsuperscript{3} Centennial Coal
CHARACTERISTICS OF MINING CONDITIONS AND SURFACE LITHOLOGY IN THE SOUTHERN COALFIELD

The collieries in the Southern Coalfield lie in the southern part of the Permo-Triassic Sydney Basin, within which the main coal bearing sequence is the Illawarra Coal Measures, of Late Permian age. The Illawarra Coal Measures contain numerous workable seams, the uppermost of which is the Bulli Seam, and it is generally this seam which has been extracted under rivers and cliffs.

The river beds and cliffs that subside from mining lie within the Hawkesbury Sandstone unit, although there are outcrops of the Wianamatta Shale Group within their catchment areas.

The depth of cover over the Bulli Seam is generally between 400 and 500 metres. The thickness of the seam is generally between 1.8 and 3.5 metres, which is usually fully extracted, using longwall mining techniques.

MINE SUBSIDENCE RELATED MOVEMENTS THAT OCCUR BEYOND THE EDGE OF LONGWALLS

The maximum observed subsidence movements occur above the extracted longwall panels. Typically, the amount of subsidence at the goaf edge is less than 50% of the maximum. Where the surface is relatively flat, the observed movements reduce with increasing distance from the goaf edge. As a general guide, they typically extend to approximately half a depth of cover from the edge of longwalls, particularly in the Southern Coalfield. Given that the typical depths of cover are between 400 and 500 metres, the limit of subsidence (as defined by a limit of 20 mm) is generally observed to be between 200 and 250 metres from the edge of longwall panels.

Where the surface includes river valleys or gorges, additional movement patterns are associated with longwall mining. Valley closure and upsidence movements are consistently observed in river valleys and gorges in the Southern Coalfield. The sides of valleys are observed to close in response to mining, with concentrations of compressive strain generally observed near the bases of the valleys. The bases of the valley are generally observed to rise relative to the valley sides, and this is termed upsidence. Similar movements have been observed in valleys even where no mining has occurred, and these movements are referred to as valley bulging and it is a natural phenomenon (Patton and Hendren, 1972), although it is understood that the process is accelerated by mining. The upsidence and closure movements are thought to occur in response to a redistribution of in-situ stress as a result of mining. Further details regarding observations, mechanisms and methods of predicting valley closure and upsidence are provided in a report by Waddington Kay & Associates (2002), now named Mine Subsidence Engineering Consultants Pty Ltd.

Closure and upsidence effects can be detected some distance from the edges of longwalls, well beyond the limit of vertical subsidence. In deep valleys and gorges in the Southern Coalfield, closure and upsidence movements have been detected more than 500 metres from the edges of longwalls, which is approximately twice the distance to the limit of subsidence in plateau areas.

In relation to vertical movements in rivers, the net vertical movement is a combination of subsidence and upsidence. If subsidence is greater than upsidence, net subsidence is observed. Conversely, the base of the river may experience net uplift if the upsidence is greater than the subsidence.

The various modes of movement reduce the further away a point is from the edge of longwalls. Depending on the proximity of longwalls to a river, upsidence can exceed subsidence beyond the goaf edge, resulting in net uplift. In the Cataract Gorge, for example, monitoring indicated that upsidence of 700 mm occurred while the area generally subsided by 500 mm, resulting in a net uplift of 200 mm.

POTENTIAL IMPACTS THAT CAN OCCUR AS A RESULT OF MINING

Rivers can experience a number of potential impacts as a result of mining, many of which have been well documented (Holla and Barclay, 2000). A summary of potential impacts is listed below.

- Fracturing in the riverbed and rockbars
- Surface water flow diversion from the surface to the shallow sub-strata
- Additional ponding, flooding or desiccation
- Additional erosion
Changes to stream alignment
Changes to water quality
Impacts on terrestrial and aquatic flora and fauna

Where mining is close to, but not directly under rivers, the changes in the gradient that occur as a result of subsidence are generally small and an order of magnitude less than the natural river gradients or cross-gradients. The potential for additional ponding, flooding, desiccation or changes to stream alignment are therefore very small. Scouring or increased erosion is of greater concern for alluvial beds or bedrock containing soft strata, but not for the relatively hard Hawkesbury Sandstone that is found in the beds of the major rivers in the Southern Coalfield.

Fracturing of rock and surface water flow diversion are the most visible and well known impacts associated with mining beneath rivers. The potential changes to water quality and, to a lesser extent impacts to flora and fauna are also largely dependent on the severity of these impacts.

Further details regarding fracturing and surface water flow diversion are discussed below.

**OBSERVATIONS OF FRACTURING BEYOND THE EDGE OF LONGWALLS**

Fractures and joints in bedrock and rockbars occur naturally during the formation of the strata, and from erosion and weathering processes, which include natural valley bulging movements. When longwall mining occurs in the vicinity of creeks and rivers, mine subsidence movements can result in additional fracturing or reactivation of existing joints. A number of factors are thought to contribute to the likelihood of mining-induced fracturing and these are listed below.

- Mining-related factors, which affect the level of ground movements that occur. These factors include among other things, the depth of cover and proximity of the mining to the river, panel width and extracted thickness.
- Topographic factors associated with the river valley, which include valley depth and steepness of the valley.
- Local, near-surface geological factors, which include bedrock lithology such as rock strength, thickness of bed strata, orientation and dip of strata, degree of cross-bedding and existing jointing.
- In-situ horizontal stresses in the bedrock.

A number of collieries in the Southern Coalfield have recorded the extent and location of fractures that have developed during and after longwall mining operations in the vicinity of rivers and gorges. The mining operations reviewed for this paper include Tower and Appin Colliery beneath or near the Cataract River, Tahmoor Colliery beneath or near the Bargo River, West Cliff Colliery beneath or near the Georges River, and Elouera Colliery beneath or near Wongawilli Creek. Other operations in the Southern Coalfield have also mined beneath or near rivers and creeks and these are the subjects of other research.

In comparison, where river beds have not been directly mined beneath, the effects of subsidence have occurred to a substantially lesser extent. Where the rivers have not been directly mined under, a smaller number of mining-induced fractures have been observed. The fractures have been observed in local, isolated areas. These fractures were noted as minor in monitoring reports, and there were no indication of changes to flow conditions in these areas. It was found that the majority of mining-induced fractures are observed where rivers are located directly above extracted longwalls.

Figure 1 shows a graphical representation of fracturing relative to distance from the nearest edge of longwall mining, which can be measured from fracture maps that have been provided by each colliery. The plot refers to the number of observed fracture sites and not the number of fractures. Substantially more mining-induced fractures have been found at each fracture site above extracted longwalls.
In addition to the above information, extensive monitoring was undertaken as longwalls approached and passed beneath the Georges River at West Cliff Colliery. Fracturing was generally noticed in each section of river after it had been directly mined beneath, although some minor fracturing or movement of existing joints occurred in front of the extraction face. These minor subsidence movements did not impact water flows or quality in the river.

The above observations appear to generally correlate with established understanding of the behaviour of sandstone under strain. Fracturing is generally considered possible if systematic tensile strains are greater than 0.5 mm/m, or compressive strains are greater than 2 mm/m. A conservative analysis of observed ground movements suggests that compressive strains due to closure are generally greater than 2 mm/m in deep valleys (greater than 50 metres) only when they are within approximately 250 metres of the goaf edge, as shown in Figure 2. Only minor fracturing has been observed within rivers within this proximity to mining.

The observation of small amounts of localised fracturing at more remote distances from extracted longwalls is understandable as the level of stress and bedrock strength varies along the length of a river. The level of existing stress in the bedrock varies depending on its position in the natural erosive cycle and the level of in-situ stress that has been imposed on it. The bedrock strength varies along the river depending on the type of rock, its layer thickness and extent of natural joints and fractures.
Fractures resulting from small subsidence movements, such as those that occur adjacent to longwall mining can occur where the bedrock is close to its elastic limit. A good example is the fracture in the bedrock beneath Broughtons Pass Weir on the Cataract River (identified in Fig 1), which occurred in rock that was already under stress as a result of the excavation and construction of the Weir, and its strength was relatively weak given that it was a thin layer of shale, rather than a layer of massive sandstone.

Given the above complexities, it is difficult to accurately predict precisely where fractures may develop in response to mine subsidence. However, monitoring of past mining experiences provides a good level of confidence for predicting the likelihood, style and extent of fracturing in rivers from longwall mining.

**OBSERVATIONS OF SURFACE WATER DIVERSION BEYOND THE EDGE OF LONGWALLS**

Mine subsidence related impacts on surface water are primarily concerned with diversion or loss of water in the following ways:

- Diversion of surface flows into subterranean flows, where water travels via fractures and joints in the bedrock into near-surface dilated strata beneath. This water generally resurfaces further downstream.
- Leakage through rockbars, where water held in ponds and pools may leak through fractures and joints in rockbars and resurface further downstream.
- Infiltration into the groundwater system, particularly where the groundwater table is lower than the surface water level of the river.
- Surface water into the mine.

In the Southern Coalfield, the main types of potential flow diversion are subterranean flows and rockbar leakages. Diversions of surface water through these mechanisms occur naturally due to erosion and weathering processes and natural valley bulging movements. Natural surface flow diversions are observed along the Cataract, Georges and Bargo Rivers in areas unaffected by mining.

Infiltration of surface water into deeper groundwater can not result unless a conduit is established for flow through to a deeper permeable horizon. Surface water loss is generally unlikely in the long term, especially where the groundwater table is higher than the surface water level of the river. Loss of surface water into the underground mine workings has not been observed in the Southern Coalfields where the depths of cover to the Bulli Seam operations is generally greater than 400 – 500 m, and the presence of the Bald Hill Claystone, which acts as an aquiclude.

Mining-induced surface flow diversion into subterranean flows occurs where there is an upwards thrust of bedrock, resulting in fracturing of the rock and redirection of surface water through the dilated strata beneath it. The water reappears downstream of the fractured zone as the water is only redirected below the river bed for the extent of the subsidence induced fracturing. This type of water loss has been observed previously in the Cataract, Georges, and Bargo Rivers and is illustrated by Figure 3.

Mining-induced surface flow diversion due to rockbar leakage occurs in a similar manner to the above mechanism, except that the rockbar is elevated above the rest of the river bed and the general water table. The rate of leakage is dependent, among other things, on the extent of horizontal fracturing within the depth of the rock bar and the water level. The rockbar leaks at a higher rate when the pool is full as there is access to all drainage paths and the water pressure is at its highest. However, as the pool level falls, the drainage rate reduces as the water pressure falls and access is restricted to drainage paths near the base of the rockbar. This type of flow diversion has been observed previously in the Cataract and Georges Rivers and is illustrated by Figure 4.
As a key component to subsidence management, collieries in the Southern Coalfield routinely undertake investigations into the location and extent of surface flow diversions during and following longwall mining operations. Studies at Tower and Appin Colliery beneath or near the Cataract River, Tahmoor Colliery beneath or near the Bargo River, and West Cliff Colliery beneath or near the Georges River have been reviewed in the preparation of this paper.
The following comments are made from these reviews:

- There are no conclusive examples of surface water flow diversion beyond the edges of longwalls in the Cataract River. There is one site near the commencing end of Longwall 405 at Appin Colliery, where the surface flow diversions of approximately 2 ML/day have been observed but limited pre-mining investigations indicated that subterranean flows occurred prior to mining.

- Some sections of the Bargo River were observed to completely drain following a prolonged period of very low flows. The furthest distance of observed surface flow diversions from the edge of the extracted longwall was approximately 125 metres. Unfortunately, there were no pre-mining investigations to differentiate between natural and mining-related flow diversions.

- Periodic monitoring of surface flows in the Georges River indicate that surface flow diversions did not begin until the longwall passed directly beneath it. Water levels in pools were not observed to fall until the longwall had passed directly beneath them by 140 to 180 metres.

The potential for noticeable or complete surface water flow diversions are not only dependent on the amount of fracturing and bed dilation, but also the magnitude of flow in the stream. A simple formula can be used to illustrate the importance of water flow.

\[
\text{Upstream Flow} = \text{AFFECTED RIVER SECTION} = \text{Downstream Flow} = \text{Diversionary Flow} + \text{Surface Flow}
\]

The formula simplifies an extremely complex system of flow conditions which vary from one section to another as a result of, for example, natural surface flow diversions and inflows from streams and other catchment areas within the affected river section. However, the formula is useful for demonstrating flow conditions of concern. If the maximum allowable diversionary flow is greater than the upstream flow, there will not be any surface flow within the impacted section of the river. If the rate of leakage through a rockbar is greater than the upstream flow, the pool will eventually drain.

Diversionary flow comprises two elements: natural diversionary flows and mining-induced diversionary flows. At present, there is relatively little information available to determine the amount of flow that is naturally diverted in streams. This is because until recently, detailed investigations of flow conditions in the river prior to mining had not been undertaken. However, post-mining investigations can estimate the total amount of natural plus mining-induced diversionary flows by observing the minimum level of upstream flow that is required to keep some surface water flowing along the river.

A good demonstration of this principle can be found by comparing the time that impacts were reported in a river with flow rates that were measured upstream. Figure 5 shows measured upstream flow rates over time in the Bargo River, as Longwalls 14 to 19 at Tahmoor Colliery were extracted. It can be seen from Figure 5 that flows were observed to remain continuous on the surface provided that flows were 2 ML/day or greater. The highest upstream flow that was recorded while discontinuous surface flows were observed downstream was 1.7 ML/day. Continuous surface flows were observed in 2004 and 2005, even during times of low flow, as the fractures had been naturally sealed by sediment.

Prior to the installation of a grout curtain in a section of the Cataract River affected by subsidence, the minimum flow required to keep the pools flowing in this section where longwalls have mined directly under the river was estimated to be 3.5 ML/day. Where rivers have not been directly mined beneath, surface flows have remained continuous during sustained environmental flows of 1.7 ML/day. Prior to rehabilitation of the area, pools in the Georges River retained water above Longwalls 5A2 to 5A4 with flows of 1.9 ML/day.
These past experiences indicate that rivers will continue to flow even if they are impacted by direct mining, provided that flows in the river are greater than 3.5 ML/day. Collieries now routinely monitor baseline flow in rivers ahead of mining to understand flow conditions and identify sections along which flows are naturally diverting beneath the river bed. As these recently implemented studies mature, the collection of appropriate data and rigorous analysis is resulting in a far greater understanding of the impacts of subsidence on streams is becoming available. This in turn is allowing for much improved confidence in impact assessments and mine planning.

**OBSERVATIONS OF IMPACTS TO CLIFFS BEYOND THE EDGE OF LONGWALLS**

Instabilities occur naturally along cliff lines due to a number of factors, including erosion and weathering, water seepage and water pressure, in-situ stresses in the bedrock, thermal expansion and contraction, changes in moisture content and the natural movement of the cliff lines.

Mining can result in differential movements along cliffs which induce additional stresses in the rockmass, and as a result, can potentially reduce the stability of cliffs. The major factors which influence the stability of cliffs, both naturally and due to mining, are summarised below:

- **Cliff geometry:**
  - Height of the cliffs
  - Length of the cliffs
  - Shape, or horizontal curvature of the cliffs
  - Slope of the cliffs,
  - Size of the overhangs or undercuttings along the cliffs
  - Overall height of the valley

- **Geotechnical factors:**
  - Type of rock
  - Jointing, anomalies and inclusions in the rock which create weaknesses
  - Rock bedding
  - In-situ stresses in the rock
• Environmental factors:  – Erosion and weathering which create unstable blocks  
– Seepage flow and water pressure  
– Heating and cooling from sun exposure  
– Changes in ground moisture content  
– Natural movement of the cliffs  

• Mining factors:  – Magnitude of subsidence, tilt, strain and curvature at the cliffs  
– Direction of tilt, strain and curvature relative to the cliffs  
– Proximity of mining to the cliffs  
– Direction of mining relative to the cliffs  
– Depth of cover at the cliffs  

The complex interaction of the above factors makes it difficult to develop a model that can predict the likelihood of cliff or rockface instability based on predictions of ground movement. It is likely that in some cases, a rockface is close to a threshold point of instability prior to mining, and even the smallest additional movement may bring forward the timing of a rockfall.

There have been few instabilities in the Southern Coalfield as a result of mining under cliffs at depths of cover greater than 400 metres. Importantly, there have been no reported instabilities beyond the edges of longwalls. In the case of the Bargo River, there have been no reported cases of instabilities and only one small instability has been reported in the Georges River.

The most well known instances of instabilities have occurred in the Cataract River, where eight instabilities have been reported. Another two instabilities were reported in the Nepean River. These instabilities occurred during mining at Tower Colliery and their locations are shown in Figure 6, where it can be seen that no sites are located beyond the extent of the extracted longwalls. A check between the dates of each reported instability with actual longwall positions also confirmed that all instabilities occurred after they had been directly mined beneath.

The potential for cliff instability beyond the goaf edge remains even though no instabilities have been reported in the Southern Coalfield beyond the mining area. It is therefore prudent to examine the consequence of an instability occurring at each individual cliff, so that the risks associated with mining close to cliffs can be safely managed.

Fig. 6 – Observed rock falls over Tower Colliery
CONCLUSION

This paper illustrates the potential impacts of mining to rivers and cliff lines in the Southern Coalfield. It is important to capture and consider the past when examining proposed mine plans and assessing the potential for impacts to occur. While some impacts might occur beyond the edges of longwall panels, the frequency and severity of impacts are substantially reduced when compared to those that have occurred where longwalls have been directly mined under these features.

A clear understanding of potential impacts of longwall mining is essential for developing relevant baseline studies, assessing potential impacts and formation of appropriate mitigation and remedial methods. This results in management plans being implemented to monitor and mitigate the identified risks without unduly sterilising coal resources.

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REFERENCES

Holla, L. and Barclay, E, 2000. Mine Subsidence in the Southern Coalfield, NSW, Australia. Department of Mineral Resources, NSW.


MANAGEMENT OF IMPACTS OF LONGWALL MINING UNDER URBAN AREAS

Daryl Kay¹, Arthur Waddington¹, Joanne Page² and Ben de Somer³

ABSTRACT: Current and proposed expansions to existing underground coal mines are occurring in closer proximity to and directly beneath urban areas. The density of built infrastructure potentially impacted by longwall mining is substantially increased in urban areas when compared with mining beneath rural areas. Management of the impacts of longwall mining in such areas requires extensive consultation with the affected community, including education and management of communities’ expectations regarding what could be expected during and after the mining period.

Approaches undertaken by Tahmoor Colliery to manage the potential impacts of mining under the township of Tahmoor are discussed. A summary of impacts that have occurred as a result of mining the first two longwalls are outlined, which are part of a series of eight longwalls that the Colliery plans to mine beneath this town. The experience gained at Tahmoor illustrates that longwall mining beneath urban areas is sustainable and can be successfully managed to mitigate the impacts on surface developments.

INTRODUCTION

The main coal seams in the Sydney Basin are deepest at Sydney and are close to the surface in the Newcastle, Southern and Western Coalfields, where the coal can be extracted at relatively shallow depths. However, as coal reserves are extracted, mines are progressively expanding to extract coal from deeper underground. This expansion is generally directed towards Sydney.

Sydney’s population is steadily growing. It is estimated that another 1.1 million people will be living in Sydney by 2031 (Department of Planning, 2005). While the State government is trying to accommodate a large proportion of these people through urban renewal and consolidation, approximately 350,000 new dwellings are proposed for the south-west and north-west of Sydney, and the Gosford/Wyong area. Some of these areas lie within proclaimed Mine Subsidence Districts.

The above trends lead to an apparent convergence of the needs of resource recovery and urban development. Proposals for new or expanded coal mines understandably raise concern and resistance within potentially affected communities; while on the other hand, developers experience caution and resistance when they propose to construct developments above untapped coal resources.

However, in reality it is feasible for underground coal mining and urban development to co-exist, particularly when mine subsidence movements are small enough that surface features remain safe, serviceable and repairable.

The challenges presented by mining under urban areas are described. Reference is made to the approach undertaken by Tahmoor Colliery to manage the potential impacts and associated risks of mining under the township of Tahmoor in the Southern Coalfield. It also includes a summary of impacts that have occurred as a result of mining the first two longwalls, which are part of a series of eight longwalls that the Colliery plans to mine beneath this town.

MINE SUBSIDENCE

Subsidence occurs as a result of underground coal extraction by longwall mining or other mining techniques. In longwall mining, a rectangular panel of coal is totally removed by longwall shearing machinery, which travels back and forth across the coalface. As the longwall steps forward, the rocks immediately above the coal seam fall

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behind it to fill the void left by the extracted coal. The mechanism progresses upwards through the layers of rock as they fall or sag into the void, resulting in subsidence at the surface.

Subsidence usually refers to vertical displacement of a point. The amount of subsidence that occurs at each point on the surface varies across the area, with greatest subsidence occurring towards the centre of the longwall, and progressively smaller amounts of subsidence beyond the edge of the longwall. The subsided area is similar in shape to a rectangular bowl.

Given that the amount of subsidence varies between points on the surface, differential movements occur as a result of subsidence in a number of ways. The first kind of differential movement is ground tilt, which is defined as the change in slope of the ground. The second kind of differential movement is ground curvature, which is defined as the rate of change in tilt and is expressed in terms of hogging curvature or sagging curvature. The third kind of differential movement is ground strain, which is defined as the rate change in horizontal movement between two points. In areas where hogging curvature occurs, the ground strains are typically tensile at the surface and the ground expands. In areas where sagging curvature occurs, the ground strains are typically compressive at the surface and the ground is compressed.

CHALLENGES PRESENTED BY MINING UNDER URBAN AREAS

Mining under urban areas presents many challenges that must be considered when assessing potential impacts.

- **High density of surface features**
  There are a high proportion of covered areas in urban environments, particularly in high density developments such as flats or units. In addition to the areas covered by building structures, a survey of 120 properties by MSEC at Tahmoor and Thirlmere revealed that only one property did not have an external pavement of some kind. In relation to publicly owned land, a large amount is covered by sealed roads. The high density of surface features in urban environments increases the chances of adverse impacts occurring, when compared to rural areas.

- **Great variety of services infrastructure**
  Urban areas are serviced by power and communications systems, potable water networks, sewerage systems, and gas reticulation pipework. Additionally, there are also many transport systems such as rail networks, local roads, footpaths, bridges and culverts. There is great inter-dependence between services. For example, if the electrical network is compromised, sewage pumping stations can lose power, which could then lead to other adverse impacts.

- **More people are potentially affected**
  This presents many challenges for the mining industry. These are discussed in more detail later in this paper.

- **Large number of public amenities**
  Public amenities provide great support to urban communities. Amenities include schools, churches, shops and shopping centres, child care centres, health services and sporting fields.

- **Large number of business and commercial establishments**
  There are many business and commercial establishments within urban areas. Mining companies are exposed to any consequential losses associated with subsidence impacts.

- **Urban areas are dynamic and continually changing**
  Studies by Australian Bureau of Statistics indicate that the Wollondilly’s population grew at a rate of 2.1 % per annum between 1991 and 2001 (WSC, 2006). A study by MSEC on the net rate of growth of houses in Tahmoor indicated that approximately 2 to 3 additional houses are being constructed in Tahmoor per month. The population growth and development of rural areas is substantially less.

- **Structures are in various conditions**
  Urban areas contain dwellings of many ages. Some dwellings are listed as items of heritage significance, while others may be only recently constructed. The existing condition of some older structures may not meet current Australian Standards, before mining occurs. Furthermore, some structures have been built
prior to the proclamation of Mine Subsidence Districts and have not, therefore, been designed to accommodate mine subsidence movements.

- **Impacts are more easily observed**
  Given the high density of surface features and the number of people who live and work in urban areas potentially affected by mine subsidence, there are more opportunities to observe any impacts that might occur. In a rural environment, for example, there is a good chance that a crack in the ground will not be noticed. However, in an urban environment, these movements may result in adverse impacts, which can be observed and reported by many people.

**CASE STUDY ON MINING UNDER TAHMOOR**

Tahmoor is located approximately 100 kilometres south-west of Sydney, within the Southern Coalfield. The town was first settled in 1820's, initially to house travellers traversing the ‘great south road’ (Stonequarry, 2005). The town is now the largest in the Wollondilly Shire, with a population greater than 4000 people (ABS, 2001). There are over 1500 dwellings in Tahmoor (ABS, 2001), and approximately 2000 sheds and other small structures. Tahmoor lies within the Bargo Mine Subsidence District, which was proclaimed in 1975.

The town is located on the former Hume Highway and was once part of the main vehicular transport route between Sydney and Melbourne. The Main Southern Railway between Sydney and Melbourne passes through Tahmoor, carrying passengers and freight at approximately half hour intervals.

The town includes two shopping centres, which are joined by a small commercial district. A turkey processing plant is also located at Tahmoor.

Tahmoor Colliery commenced operations in 1979 and holds mining leases that include Tahmoor and some parts of neighbouring Thirlmere and Picton. The mine employs approximately 400 people.

Tahmoor Colliery has previously mined under many houses and other structures. It commenced mining under the urban area of Tahmoor in early 2005 with Longwall 22. It is currently mining Longwall 23 and a further six longwalls are planned to extract coal beneath the extent of the urban area. The extracted coal seam is approximately 400 to 500 metres beneath the surface, with subsidence expected to reach a maximum of 750 mm after the extraction of Longwall 23.

![Fig. 1 - Location of Tahmoor and previous and future mining](image-url)
METHODS EMPLOYED TO MANAGE IMPACTS AND RISKS AT TAHMOOR COLLIERY

A number of measures are being employed by Tahmoor Colliery to manage the impacts and risks associated with mining under the urban areas.

- **Identification and characterisation all surface features that may potentially be affected**
  
  As part of its assessment, studies of all surface features have been undertaken. This included identification of all houses and other structures, public amenities and commercial and business establishments. Letters are sent to residents requesting them to check on the accuracy of the surveys and advise of any changes.

  Information on all services has been collected from infrastructure owners and meetings were held to understand how each service operates.

  It is also recognised that Tahmoor is growing in population and size and all information collected only represents a snapshot in time.

- **Assessment of likely impacts**
  
  Greater confidence in predicting likely impacts can be achieved by undertaking detailed impact assessments. This has been a complex task given the density of features above the longwalls. For example, there are over 39 kilometres of sewer pipes in Tahmoor and predictions of subsidence were conducted along every length of sewer in Tahmoor, to determine whether the grades of any lengths of sewer were likely fall below self-cleansing grade.

  Impact assessments are then made based on increased subsidence predictions, so that the sensitivity of each surface feature could be understood.

- **Consultation with the community**
  
  The Colliery continues to consult with the community on many levels. It has been found that this consultation has greatly assisted the Colliery in identifying and characterising surface features, understanding how sensitive they might be to mine subsidence, and monitoring and reporting impacts that occur.

- **Identification of potentially unstable structures**
  
  It is difficult to identify potentially unstable structures without entering private properties. Tahmoor Colliery has addressed this risk in two ways. Firstly, it invites residents who live in older homes, particularly those that were constructed prior to the declaration of the Bargo Mine Subsidence District, to an inspection by the Colliery prior to mining. Secondly, it has sent letters asking all residents to advise them of any concerns that they may have in relation to the stability of their structures. Thirdly, the colliery conducts home visits to residents prior to mining, to generally look around the property, take photographs of any potential issues and arrange for additional monitoring during mining, if required.

- **Monitoring ground movements and impacts**
  
  Tahmoor Colliery has installed an extensive network of ground survey marks within the urban area. The design of the network was discussed and developed through consultation with the Department of Primary Industries Minerals, the Tahmoor Colliery Community Consultative Committee, the Mine Subsidence Board and the general public through open days. The ground survey network allows the Colliery to periodically check whether subsidence is developing as predicted and identify any areas where irregular and potentially damaging movements might be occurring. If adverse impacts occur, the survey network allows the Colliery to quantify the subsidence movements and check whether these movements are irregular.

  The Colliery also conducts routine visual inspections of surface features for impacts. These are mainly conducted within the ‘active subsidence zone’, which is over an area that is defined by a distance of 150 metres in front and 450 metres behind the longwall face, within the predicted limit of subsidence.

  In relation to services infrastructure, the colliery and infrastructure owners have engaged in a number of monitoring programs that are specific to the needs of each type of infrastructure. There are also automated monitoring systems that are already operated by infrastructure owners that can detect whether impacts are occurring.
• **Risk assessments and management plans**

Risk assessments and management plans have been developed for all surface features that may be potentially affected by mine subsidence. Where possible, the risk assessments and management plans have been produced in consultation with stakeholders, such as infrastructure owners.

• **Close liaison with Mine Subsidence Board**

The Mine Subsidence Board (MSB) is charged with the responsibility of repairing any damage to properties as a result of mine subsidence. The community often reports impacts to the Colliery or the MSB and it is important that the colliery and the MSB maintain a close working relationship so that both parties are knowledgeable on all the impacts that occur during mining.

**MANAGING COMMUNITY EXPECTATIONS AT TAHMOOR COLLIERY**

The concept of mining under houses can understandably create fear and concern for some residents, particularly if they have no experience of mine subsidence. Several approaches have been adopted by Tahmoor Colliery to explain to the community how mine subsidence develops, what impacts might occur, and how these impacts will be managed. This is a very time-consuming but important process.

The initial community consultation commenced when the Colliery applied for development consent to mine. A commission of inquiry was undertaken as part of this process. Following approval to mine beneath the town, Tahmoor Colliery continued to develop their mine plans. These plans were discussed with the Tahmoor Colliery Community Consultative Committee (TCCCC), which was set up in accordance with the conditions of development consent. Prior to mining the first longwall beneath Tahmoor, the Colliery increased the level of communication with the community.

The approaches adopted by Tahmoor Colliery are listed below.

• **Undertake conservative predictions and impact assessments**

Tahmoor Colliery and MSEC have adopted a conservative approach to predicting subsidence and assessing impacts. This reduces the likelihood of under-stating the predicted impacts. For example, predictions for each structure have been made by predicting the maximum subsidence, tilt and strain within a 20 metre radius around each structure.

• **Undertake detailed predictions and impact assessments**

By undertaking detailed subsidence predictions, the Colliery is able to provide residents with predictions for their own structures. Individual assessments provide some comfort to concerned residents. This is particularly helpful for residents that live beyond the extent of mining and are expected to experience only small movements.

• **Community Open Days**

A number of advertised open days are held by the Colliery through the year. The Open Days allow members of the community to directly meet Colliery representatives and its consultants. The Mine Subsidence Board is also present on Open Days to answer questions. The information exchanged at Open Days also assist the Colliery, as members of the community sometimes provide information about particular surface features or impacts that the Colliery might not have been aware of.

• **Tahmoor Colliery Community Consultative Committee**

This committee meets at regular (bi-monthly to quarterly) intervals. It allows the Colliery to present information to the committee and receive feedback. The committee is committed to ensuring that the concerns of the community are well understood by the Colliery. Many of the members have been part of the committee for several years, and this allows for informed discussion to take place.
• **Letters to residents**

The Colliery sends many letters to community advising of imminent longwall mining in their area. These letters invite residents to contact the colliery about any concerns that they might have and to remind them to organise a pre-mining inspection if they wish to do so. Some letters may be specifically targeted to residents if the Colliery wishes to conduct its own inspection of the property, which include all public amenities, old houses and houses that have been assessed at a higher level of risk of impact. Letters are also sent to residents just before the longwalls mine directly beneath their homes. These letters again invite residents to arrange a pre-mining inspection and includes a fridge magnet with key phone numbers in the event of an emergency or impact occurring.

By continuing to engage with residents at each stage of mining, the Colliery is able to find new residents who might not have been aware that mining was taking place.

• **Individual meetings with residents**

Many members of the community prefer to meet with Colliery representatives face to face. The Colliery has held many individual meetings with concerned residents to explain how mine subsidence develops and what the impacts might be. This is a time consuming but rewarding process for residents and the Colliery.

• **Newspaper advertisements**

The Colliery places advertisements in the newspaper from time to time to advise the community at large about recent mining applications.

• **Weekly reporting**

The Colliery provides regular updates on the progress of mining in the area. This is conducted mainly by group email to any member of the community who wishes to be regularly informed. The updates advise the current position of the longwall and what impacts have been observed during the past week.

• **Prompt response to reported impacts**

While this is traditionally the role of the MSB, the Colliery also responds quickly to impacts that are reported by the community. Once an impact is reported, the Colliery also checks neighbouring properties to see whether the incident is localised or part of a larger potential issue.

• **Ongoing monitoring if impacts occur**

Where impacts have been reported, the Colliery offers to continue monitoring the property for further impacts. This offer is in addition to those provided by the Mine Subsidence Board, who also monitors the property as mining continues.

The Mine Subsidence Board also plays a very important role in managing the expectations of the community. The MSB’s concerted efforts to quickly respond to residents’ concerns, particularly where they relate to emergency repairs to doors, gates or service pipes, have greatly assisted the community in coping with any inconvenience that may have occurred as a result of mine subsidence.

**IMPACTS OBSERVED TO DATE AT TAHMOOR COLLIERY**

A comparison between predicted and observed impacts to surface features, following completion of Longwall 23A, is summarised in Table 1. It can be seen that the impacts to surface features have been relatively minor. The predicted and observed impacts to surface features compare reasonably well, with the exception of locations where non-systematic movements have occurred.
Table 1 - Summary of Predicted and Observed Impacts

<table>
<thead>
<tr>
<th>SURFACE FEATURE</th>
<th>PREDICTED IMPACTS</th>
<th>OBSERVED IMPACTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>NATURAL FEATURES</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Myrtle Creek</td>
<td>Potential cracking in creek bed.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td></td>
<td>Potential surface flow diversion.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Potential reduction in water quality during times of low flow.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Potential increase in ponding.</td>
<td></td>
</tr>
<tr>
<td>Aquifers or Known Groundwater Resources</td>
<td>See Farmland and Facilities - Wells and Bores</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td>Natural Vegetation</td>
<td>No impacts anticipated.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td><strong>PUBLIC UTILITIES</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Railways</td>
<td>Ground movements unlikely to impact operation of railway.</td>
<td>No impacts observed. Track was de-stressed as a precaution.</td>
</tr>
<tr>
<td>Roads (All Types)</td>
<td>Minor cracking and buckling may occur in isolated locations.</td>
<td>Impacts observed on road pavements in 3 locations. Impacts observed on concrete kerbs and gutters in 8 locations. Impacts have been minor and include cracking and buckling.</td>
</tr>
<tr>
<td>Water Pipelines</td>
<td>Minor impact to pipelines, particularly older cast iron pipes with lead joints.</td>
<td>Two leakages observed at connection to consumer lines during LW 22. Observed frequency of incidences similar to those in areas not affected by mine subsidence.</td>
</tr>
<tr>
<td>Gas Pipelines</td>
<td>Ground movements unlikely to adversely impact pipelines.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td>Sewerage Pipelines</td>
<td>Mining induced tilt may reduce gradient of some pipes to less than that required for self-cleansing.</td>
<td>Changes in tilt have occurred within predicted range. Observed frequency of incidences similar to those in areas not affected by mine subsidence.</td>
</tr>
<tr>
<td>Electricity Transmission Lines or</td>
<td>Ground movements unlikely to adversely impact electrical infrastructure.</td>
<td>One local feed line was loosened.</td>
</tr>
<tr>
<td>Associated Plants</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Telecommunication Lines or Associated</td>
<td>Ground movements unlikely to adversely impact telecommunications infrastructure.</td>
<td>Air leaks observed in old lead cables in one location during LW 22 and two locations during LW 23A.</td>
</tr>
<tr>
<td>Plants</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>PUBLIC AMENITIES</strong></td>
<td>Negligible impacts predicted for all public amenities.</td>
<td>Category 0 (hairline) crack to plasterboard ceiling of a child care centre.</td>
</tr>
</tbody>
</table>
**Table 1 - Summary of Predicted and Observed Impacts (continued)**

<table>
<thead>
<tr>
<th>SURFACE FEATURE</th>
<th>PREDICTED IMPACTS</th>
<th>OBSERVED IMPACTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FARMLAND AND FACILITIES</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Farm Buildings or Sheds</td>
<td>Negligible impacts predicted for all farm buildings and sheds.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td>Fences</td>
<td>No impact assessment was provided.</td>
<td>Impacts to fences or gates observed on 17 properties (1 rural, 16 urban).</td>
</tr>
<tr>
<td>Farm Dams</td>
<td>Potential cracking and leakage. Ground movements unlikely to result in overflowing or reduction in dam capacity.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td>Wells or Bores</td>
<td>Potential differential horizontal movements.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td><strong>INDUSTRIAL, COMMERCIAL &amp; BUSINESS ESTABLISHMENTS</strong></td>
<td>Negligible impacts predicted for all business and commercial establishments.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td><strong>PERMANENT SURVEY CONTROL MARKS</strong></td>
<td>Ground movement predicted at identified survey marks.</td>
<td>Ground movement occurred.</td>
</tr>
<tr>
<td><strong>RESIDENTIAL ESTABLISHMENTS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Houses</td>
<td>Tilt Impact Category B for 8 houses due to systematic mine subsidence movements. Strain Impact Category 1 for 152 houses and Category 2 for 2 houses due to systematic mine subsidence movements. Potential for non-systematic movement to occur. All structures expected to remain safe, serviceable and repairable during and following mining.</td>
<td>Some impacts observed for 33 houses, although most are negligible to very slight (sticky doors, minor cracks to internal finishes) 2 houses require adjustment to some roof gutters (Tilt Impact Cat B). 2 houses with Category 1 crack, 4 houses with Category 2 crack, and 3 houses with Category 3 crack. Houses with greater impacts had experienced non-systematic mvmts. All structures were safe, serviceable and repairable during and following mining.</td>
</tr>
<tr>
<td>Retirement or Aged Care Villages</td>
<td>Negligible impacts predicted for Macquarie Grove Retirement Village.</td>
<td>No impacts observed.</td>
</tr>
<tr>
<td>Associated Structures such as Workshops, Garages, On-Site Waste Water Systems, Water or Gas Tanks, Swimming Pools or Tennis Courts</td>
<td>Potential impact to pipes connected to in-ground septic tanks. Negligible impacts predicted for non-residential domestic structures, including swimming pools and tanks.</td>
<td>No impacts observed to in-ground septic tanks or other tanks. Tilt impact observed at 1 pool. Tilt impact observed at 1 clothesline. Crack to 1 masonry retaining wall.</td>
</tr>
<tr>
<td>External Residential Pavements</td>
<td>No impact assessment was provided.</td>
<td>Impacts to pavements observed on 16 urban properties.</td>
</tr>
</tbody>
</table>

A total of 547 houses and public amenities are located within the predicted limit of subsidence for Longwalls 22 and 23A. It was predicted that 154 houses might experience impacts. Impacts have been observed for 33 houses and one public amenity at this stage. This represents a ratio of 6%. All houses have remained safe, serviceable and repairable throughout the mining period.
Figure 2 shows the location of all properties that have experienced impacts to their houses or other structures such as sheds, fences and gates. It can be seen that the majority of impacts have occurred to properties that lie directly above extracted longwalls, although some of these properties first reported impacts before the longwall passed directly beneath them.

![Diagram showing property locations](image)

**Fig. 2 - Location of properties that have experienced impacts after Longwall 23A at Tahmoor Colliery**

**CONCLUSION**

Mining beneath urban areas is a challenging exercise and has the potential to inconvenience many people. It requires careful planning and management. However, as shown by the approach undertaken at Tahmoor, it is feasible for underground coal mining and urban development to co-exist, particularly when mine subsidence movements are small enough to result in few impacts on the surface. It is important that the mining industry learn from the experience at Tahmoor, as it is likely that mining will occur under other urban areas in the future.

**ACKNOWLEDGEMENTS**

The authors of this paper would like to thank Centennial Coal Tahmoor for providing information relating to mining under urban areas.

**REFERENCES**

STERILISATION OF COAL RESOURCES IN THE SOUTHERN NSW COALFIELDS: THE CFMEU PERSPECTIVE

Graham White

INTRODUCTION

The Construction, Forestry, Mining and Energy Union covers workers in several major industries, including coal mining. It represents an amalgamation of many trade-based and industry unions. One of these was “the Miners’ Federation” (ACSEF - Australasian Coal and Shale Employees’ Federation) that was formed in 1915. Union history shows sporadic coal unionism – including southern New South Wales – dating back to the 1850s. Within the coal industry the CFMEU represents the overwhelming majority of production and maintenance workers – over 15,000 across Australia. As a result of a high level of unionisation, and a history of strong union campaigning, coal mineworkers have wages substantially above average weekly earnings (around double) and enjoy better annual leave, sick leave and long service leave than in other industries. This means that coal mining jobs are particularly valuable jobs to have in a community – the income and expenditure from coal mining jobs has a greater local benefit than other lower-paying jobs.

THE ECONOMIC SIGNIFICANCE OF COAL MINING

For many decades coal has been Australia’s major export earner and has been the foundation for Australia’s ability to trade with the rest of the world. In the financial year 2005-06 it is estimated that coal will earn over $25 billion for Australia – a figure that is a country mile anead of any other mining, manufacturing or service industry (Australian Bureau of Agricultural and Resource Economics, 2006).

The southern NSW coal field produces mostly coking coal – the more valuable coal used in the iron and steel industry. Significant amounts are exported through Port Kembla, and much is used by the Bluescope steelworks at Port Kembla and the OneSteel plant at Whyalla in South Australia – the foundations of Australia’s iron and steel industry.

The Port Kembla Coal Terminal exported 9.2 Mt of mostly coking coal in 2005 – valued at around $122 per tonne. Another 5 to 6 Mt of coal was sold to Australian steel mills at similar prices. The total value of the coal mined in the NSW southern coal fields is between $1.6 billion and $2 billion.

There are around 2,300 people employed in the southern coal mines, with average earnings of around $2,150 per week, or $112,000 per year. Mineworkers live locally and spend locally – meaning most of the $258m they earn directly benefits the local economy. This is in addition to the substantial sums spent by the mining companies on local procurement of equipment, goods and services.

Every coal mining job directly creates another two to three jobs through demand for goods and services. Indirectly, many more people are dependent on the coal industry.

PROSPECTS FOR COAL IN SOUTHERN NSW

Due to the rapid economic growth in Asia – especially China – there is now a period of sustained demand for Australian minerals on world markets – especially coal, and especially premium coking coal like that produced in southern NSW. Coal prices are far higher than there were in the 1990s, and most industry analysts see a sustained minerals boom. All booms do come to an end, and there may well be economic upheaval and uncertainties in Asia. But there is little doubt that China is an economic powerhouse that has a long way to go over the next two to three decades.

In southern NSW, old mines have been re-opened, new mines have been developed (notably Dendrobium) and the major producer, BHP Billiton, has stated plans to increase production significantly. The southern NSW coal industry is not an industry that is limping or ailing; it has a good future provided it has access to coal resources.

1 Vice-President, CFMEU Mining and Energy South West District
The climate change problem is the principal issue that clouds the future of coal. The CFMEU is reasonably certain that coal use for power production will become a low-emissions technology within a generation, through the use of carbon capture and storage (CCS). In the longer term, coal may even be the basis for the "hydrogen economy" – where coal is the feedstock for energy technologies that are both zero-emission and ultra-reliable. In this context it would be extremely foolish for the region of southern NSW to have its economic prospects curtailed through poor land-use planning that arbitrarily excludes coal mining.

COMPETING AND CO-EXISTING LAND USES

Coal mining has co-existed with limited housing and urban development for generations. For it to work successfully, people living in mining areas need to accept that there are occasional impacts from mining – notably subsidence.

Recent experience shows that people may buy housing in designated mining subsidence areas but pay little attention to that statement in the sale contract for their property. Further, some people buy property that is lower in value because of its designation as subject to subsidence or to a mine lease, and then seek to improve the value through lobbying for restrictions on mining.

These are problems that can be managed with better information and public education. Further, and especially in the case of the southern coal fields, the point needs to be made that coal mining and coal leases have preceded proposals for urban development.

Coal mines have an historical and legal right to be in southern NSW. New urban development – often project housing that will be little more than dormitory suburbs for Sydney – should ensure that it is compatible with coal mining rather than vice-versa. It should either be built to cope with potential subsidence and mine surface infrastructure, or postponed until mining is complete.

With respect to national parks, the CFMEU has long been of the view that the blanket prohibition on mining underneath such parks is excessive. Underground mining proposals that can ensure with very high certainty that they can preserve surface ecological systems and values should be allowed within national parks. The mine owner would have to accept the responsibility to “make good” where unforeseen ecological impacts do occur. This is already the practice in the NSW southern coalfield. BHP Billiton has already undertaken substantial projects to restore water flows and waterholes (eg in the Cataract River and Marhnyes Waterhole, respectively) where unexpected losses have occurred.

Mining under national parks is not a proposal that the CFMEU promotes strongly. It is raised to make the point that much current restriction on mining that results in resource sterilisation is without sound scientific justification. Existing past and current poor practice on land-use planning should not be extended through further restrictions on access to coal.

Coal mining is a temporary land-use; urban development is far more long term. They should be planned to co-exist; where that is not feasible then coal mining as a highly valuable temporary use should be prioritised.

THE COAL STERILISATION PROBLEM

Research more than a decade ago (Coal Resources Development Copmmitee, 1994) showed that almost half of all coal resources in NSW are already sterilised – they are locked up under national parks, urban development or other land-uses that prevent mining. Another third is difficult to access economically unless coal prices are very high. This leaves only about 20 percent of resources available to mining, and this amount is also under threat due to the establishment of more national parks, urban sprawl and infrastructure projects. The CFMEU is acutely aware that new urban developments in the Wilton, Condell, Cawdor and Menangle Park areas threaten the prospects for coal mining in that area.
CONCLUSION

CFMEU policy and advocacy on sterilisation of coal resources has the following basis:

- Coal mining is one of many land uses and it is inevitable that conflict will arise over whether other land uses can co-exist with coal mining. Where co-existence is not possible, there needs to be rules that determine which land-use is preferable at a given point in time. In many cases sequential land-use is possible,
- Coal is a valuable commodity and coal mining is a major source of investment, revenue, export earnings and jobs,
- Coal mines and coal mining jobs generate substantial economic flow-on benefits for the regions in which they are based. Many people not employed in the coal mining industry directly or indirectly rely on coal mining for at least part of their livelihood,
- Particularly in southern NSW, coal mining has a long and proud history and is an integral part of the region’s identity, culture and economy. It has a right to be there,
- Coal mining can often co-exist with other concurrent land uses such as housing and major infrastructure (eg roads) but all stakeholders must learn to live with each other, which includes making allowances for minor impacts. (For example, housing must be built and/or repaired to cope with the effects of planned subsidence),
- Substantial amounts of the coal resources of NSW have already been sterilised by land-use planning decisions that preclude coal mining. Further sterilisation has the potential to severely limit the future prospects of the coal industry, with negative implications for all, and
- It is important that further sterilisation of coal resources not occur without strenuous efforts being made to enable either co-existence with other land-uses, or sequential land-use that enables coal resources to be extracted prior to other activities.

REFERENCES

Coal Resources Development Committee, 1994. Effects of land use on coal resources, Report for NSW Minister for Mines
ABSTRACT: Results of an investigation into ground vibrations carried out in an open pit coal mine in New South Wales has been analysed. As this mine is located in a sensitive area in regard to potential damage from ground vibrations, severe restrictions were imposed on the blasting operations. In this study 44 sets of recorded experimental blast data have been analysed. Two valuable equations have evolved from the data when two types of explosives were used. A comparison between the effects of two explosive types, namely ANFO and slurry, on ground vibration is presented. It has been shown that the intensity of ground vibration is greater for slurry in short distances but becomes same in a specific longer distance. It was also found that in long distances the intensity of vibration is greater using ANFO than slurry.

Keywords: Blasting, ground vibration, coal mine, particle velocity, explosive type.

INTRODUCTION

The understanding of blast induced ground vibrations is of prime importance in controlling environmental problems. The explosive type is one of the key factors in determining the intensity of ground vibration. The quantification of the effect of this parameter on vibration has been discussed in detail elsewhere (Hossaini and Sen, 2004).

Cumnock South Open Cut Coal Mine is located approximately 35 km north of the town of Singleton in the Hunter Valley Coalfield in New South Wales, Australia (Figure 1). The mine site is adjacent to the Howick Open Cut Coal Mine, bounded by the New England Highway to the North, the Pacific Power Liddell to Tomago 330 kV transmission line to the South, the Coal and Allied overland conveyor to the East, and Pikes Gully Road to the West.

Due to the mine’s close proximity to the Pacific power transmission lines and a number of road bridges (Figure 2), it was of utmost importance that the blast vibration level resulting from blasting be maintained at a level acceptable to the limitations imposed.

The maximum allowable peak particle velocity, which is defined as the vector sum of the three orthogonal velocity components (Konya, 1990), imposed at Cumnock South mine was 25 mm/s for the pikes Gully Bridge, C&A Bridge, any steel transmission towers of the 330 kV transmission line, and 50mm/s for the wooden transmission towers (Walker, 1996).

Experimental blasts using two types of explosive, namely ANFO and Slurry, were monitored in order to investigate and to implement the most reliable blasting method, which would not produce any environmental problems.

In this investigation, a series of monitored experimental data was analysed. The best fitting equation for vibration prediction was established for each explosive type, through which the maximum instantaneous charge can be calculated with very high levels of confidence. A comparison of the performance of these two types of explosives from the vibration point of view was also conducted.

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1 Faculty of Mining Engineering, University of Tehran
2 Faculty of Engineering, University of Wollongong
Fig. 1 - Location of Cumnock South Coal Mine in New South Wales.

GENERAL MINE DESCRIPTION

Cumnock coal is centred in the upper Hunter coalfield and forms part of the larger Permian aged coalfield known as the Sydney Basin. The Sydney Basin comprises sedimentary rocks such as conglomerate, sandstone and shale inter-bedded with many coal seams. Several clay stones of volcanic origin occur within the sequence and, due to their consistency, are used as major stratigraphic horizons. The coal resource is composed of nine seams that dip uniformly at approximately 4° to the South East. The mining activity commenced near the sub crop in the West and moved progressively down dip to the South East in 50 m wide strips (Robinson, Hagan and Tucker, 1995). The mine produced approximately 1.1 million tonnes per annum of coal and moved about 7 million cubic metres of overburden a year through a truck and shovel (hydraulic excavator) operation.

Fig. 2 - Monitoring locations
The blasting operation was conducted in the overburden. The overburden and inter-burden consisted mostly of siltstone and medium to thickly bedded sandstone. This stratification is free of significant joints or bedding planes and medium in strength. The average un-confined compressive strength was around 45 MPa.

As Cumnock South Open Cut Coal Mine is located in an area with many potential complaints about damage due to ground vibration, strict restrictions were imposed on the blasting operation. To implement a reliable shock reduction method, experimental blasts had been carried out in order to minimize the environmental problems (Hossaini and Sen, 2006).

The outline of drilling and blasting design at Cumnock South mine was as follows:

The bench heights were 10 m, 20 m or 28 m where in some cases two passes were required in order not to exceed the vibration limitation. The hole diameter used was 130 mm for the holes drilled for creating rock buffer and 187 mm for normal blasts. The initiation sequence was such that it progressed away from the sensitive area.

Blast vibrations were monitored using three Blastronic’s Micro monitors at 5 locations shown in Figure 2. The monitoring points were positioned at sufficient distance from the structures to avoid undue vibration influence from the structures.

Cumnock South Cut operation was suspended in the late 90's. The data processed in this investigation relates to that era and not to the current project.

PEAK PARTICLE VELOCITY ANALYSIS

Data from 21 shots using ANFO (Table 1) and 23 shots using slurry explosive (Table 2) have been analyzed. When slurry was used, the explosive weight was converted into its ANFO equivalent, appearing in column 4 of Table 2 as (Maximum Instantaneous Charge) MICe (where subscript “e” stands for equivalent). As seen from Tables 1 and 2 the peak particle velocity (ppv) of ground vibration decreased rapidly as distance from the blast center to the survey station increased. The ppv decreased from an average of 37.9 mm/s at 42 m to 0.49 mm/s at 873 m for ANFO and from 32.07 mm/s at 93 m to 2.72 mm/s at 851 m for slurry.

Because the type of ground was assumed to be uniform in this study, it is reasonable to expect that any alteration in the ground vibration would be due to different explosive types as the only variable in this study.

The following scaled distance empirical equation originally proposed by US Bureau of Mines (Dowding, 1996) has been used for prediction of peak particle velocity:

\[ v = k \left( \frac{D}{\sqrt{Q}} \right)^a \]  \hspace{1cm} (1)

Where \( v \) is peak particle velocity (mm/s), \( D \) is distance (m), \( Q \) is the maximum instantaneous amount of explosive charge (kg), \( \frac{D}{\sqrt{Q}} \) is scaled distance (m/kg \(^{0.5}\)) and \( k \) and \( a \) are normally called site specific parameters.

Applying non-linear regression, to both groups of the data, the best values of parameters \( k \) and \( a \) are found for Equation (1), in each case, with excellent levels of correlation. The analysis was carried out by Microsoft Excel for XP Windows.

In the following sub-sections, the criterion is assessed against both groups of the data and the results are discussed individually for each case.
Table 1 - Ground vibration measurements in standard blasts using ANFO

<table>
<thead>
<tr>
<th>Event No</th>
<th>Distance (m)</th>
<th>MIC(Q) (Kg)</th>
<th>PPV (mm/s)</th>
<th>Scaled Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>747.00</td>
<td>218.90</td>
<td>1.90</td>
<td>50.49</td>
</tr>
<tr>
<td>2</td>
<td>733.00</td>
<td>280.31</td>
<td>1.50</td>
<td>43.78</td>
</tr>
<tr>
<td>3</td>
<td>700.00</td>
<td>228.80</td>
<td>1.60</td>
<td>46.28</td>
</tr>
<tr>
<td>4</td>
<td>873.00</td>
<td>40.00</td>
<td>0.49</td>
<td>138.03</td>
</tr>
<tr>
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<td>196.00</td>
<td>241.21</td>
<td>26.40</td>
<td>12.62</td>
</tr>
<tr>
<td>6</td>
<td>207.00</td>
<td>233.17</td>
<td>19.72</td>
<td>13.56</td>
</tr>
<tr>
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<td>629.00</td>
<td>74.33</td>
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<td>72.96</td>
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<tr>
<td>8</td>
<td>640.00</td>
<td>218.90</td>
<td>1.80</td>
<td>43.26</td>
</tr>
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<td>9</td>
<td>660.00</td>
<td>280.31</td>
<td>3.10</td>
<td>39.42</td>
</tr>
<tr>
<td>10</td>
<td>620.00</td>
<td>228.80</td>
<td>2.30</td>
<td>40.99</td>
</tr>
<tr>
<td>11</td>
<td>42.00</td>
<td>30.00</td>
<td>39.09</td>
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<td>12</td>
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<td>80.00</td>
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<td>110.00</td>
<td>46.08</td>
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</tr>
<tr>
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<td>271.61</td>
<td>39.90</td>
<td>9.28</td>
</tr>
<tr>
<td>15</td>
<td>691.00</td>
<td>241.21</td>
<td>2.93</td>
<td>44.49</td>
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<tr>
<td>16</td>
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<td>25.00</td>
<td>36.85</td>
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<td>146.94</td>
<td>10.80</td>
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</table>

Table 2 - Ground vibration measurements in standard blasts using slurry

<table>
<thead>
<tr>
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<th>MIC(Q) (Kg)</th>
<th>MICe (kg)</th>
<th>PPV (mm/s)</th>
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<td>551.78</td>
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<tr>
<td>3</td>
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<td>248.90</td>
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</tr>
<tr>
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<td>820.00</td>
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</tr>
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<td>4.00</td>
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Vibration due to ANFO

Equation (1), was applied to 21 pairs of the data relating to the shots where ANFO was used. The best fit structure of the equation has been established between peak particle velocities and scaled distances with best possible values of coefficients of correlation (R) as follows.

\[
v = 1269.9 \left( \frac{D}{\sqrt{Q}} \right)^{-1.6628} \quad (R=0.9924)
\]

Figure 3 represents the established equation along with the data.

![Fig. 3 - PPV versus scaled distance for blasts using ANFO](image)

Vibration due to Slurry

Equation (1) was applied to 23 sets of the data relating to the shots where slurry explosive was used. Non-linear regression was carried out the same way as for ANFO. The following best fit forms of the equations has been established between peak particle velocities and scaled distances:

\[
v = 2239.3 \left( \frac{D}{\sqrt{Q}} \right)^{1.838} \quad (R=0.959)
\]

Figure 4 shows the established equation along with the data.

![Fig. 4 - PPV versus scaled distance for blasts using slurry explosive](image)
Attenuation of vibration

Attenuation trend of the peak particle velocities are presented in Figures 5 and 6 for the two types of explosives. Two best fitting equations describing the attenuation due to distance have been established and these are as follows:

For ANFO: \[ v = 51.538e^{-0.0049D} \] (R=0.962) \hspace{1cm} (4)

For Slurry: \[ v = 86.119e^{-0.006D} \] (R=0.914) \hspace{1cm} (5)

Fig. 5 - Attenuation trend of vibration for ANFO

Fig. 6 - Attenuation trend of vibration for Slurry

COMPARISON OF GROUND VIBRATIONS VALUES

Figure 7 compares the magnitudes of ground vibrations for slurry and ANFO. In Figure 8 the rates of vibrations for these two types of explosives are plotted against distances. The values of ppv shown in this figure have been back calculated from equations 4 and 5 which were obtained for attenuations of vibrations as shown in Figures 5 and 6.
As seen in Figure 8, the intensity of vibration is much greater for slurry explosive. As the monitoring distance increases, both types of explosives (viz. ANFO and slurry) become similar and are virtually the same at long distances. In this study this phenomenon was evident at around 460 meters. This is because with the increase in distance the intensity of vibration is attenuated and dies down in far places. When very long distances are concerned, regardless of the type of explosive the ground vibration becomes weaker and weaker, and is finally untraceable.

An interesting point has emerged: in distances over 500 meters the intensity of vibration due to ANFO is greater than that of slurry. This can be interpreted as the effect of frequency. The waves with lower frequencies normally last longer and are more effective over long distances. Since the monitoring of vibration eaves did not record the frequencies in that period this hypothesis cannot be substantiated with any degree of certainty. However, it is likely that the waves produced by ANFO are mostly of lower frequencies than those produced by slurry. This can be considered as the reason for the higher ratio of ANFO to slurry proportion of vibration intensity in long distances.

![Fig. 7- Comparison of vibration intensity for ANFO and Slurry](image1)

![Fig. 8 - Rate of vibrations of ANFO to Slurry for various distances](image2)
CONCLUSIONS

The results of this investigation can be summarised as follows:

- The explosive type may significantly affect the intensity of ground vibration.
- In short distances, the intensity of vibration produced by slurry is much greater than that produced by ANFO.
- As the distance increases, the intensity of vibration grows closer for both types of explosives.
- At a specific distance both explosives produced similar vibration values.
- Beyond a specific distance the vibration becomes higher for ANFO than for slurry.
- The role of frequency is likely to effect the variation of vibration intensity at long distances.

ACKNOWLEDGMENTS

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REFERENCES

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