2003

Proceedings of the 2003 Coal Operators' Conference

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PROCEEDINGS OF THE 2003
COAL OPERATORS’ CONFERENCE

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FOREWORD

The underground coal operators conference series held annually in Wollongong has been recognised as the main form of the exchange of ideas between the mine operators, engineers and researchers in the diverse field of coal mining technology. For the last five years the conference addressed a variety of issues, focusing primarily on underground ground control and mine safety. In order to increase participation, the attention has now been drawn to addressing various issues in addition to ground control and the theme of Coal2003, Longwall Environment demonstrates the true interest of the conference in promoting high output longwall operation. This year the conference is preceded by a halfday workshop on mine subsidence.

The COAL2003 Conference has been generously supported by the following organizations:

- BHPBilliton, Illawarra Coal
- Roadway Reinforcement Services
- Seedsman Geotechnics Pty Ltd
- Ground Consolidation
- Fosroc Mining Pty Ltd

Our thanks go to the authors who have accepted the invitation to contribute and present papers at this conference. They represent a cross section of the developments in the coal and manufacturing industries in Australia, Germany, and The United States of America.

The Aus.I.MM Illawarra Branch and the organising committee are extremely grateful to all the above for their support.

The Organising Committee also extends appreciation to James Cook and his colleagues at the Wollongong Union Centre for the management and registration of the conference, Bruce Robertson for his assistance in Audio Visual maintenance, Leonie McIntyre of the Faculty of Engineering for type setting the Conference Proceedings, Gordon Nolan for assisting in designing of the proceedings cover and the University of Wollongong Printery for printing copies of the proceedings.

Naj Aziz, Associate Professor (Editor)
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PREFACE

The Illawarra Branch of the Australasian Institute of Mining and Metallurgy has had considerable success in organizing conferences, symposia and seminars over a number of years.

This fourth, in a series of Coal Operations conferences, is a joint collaboration between the Institute, the University of Wollongong and the Mine Managers’ Association of Australia.

The organizing committee, which represents each of the collaborators, has brought together a group of speakers who represent a broad range of interests in the coal industry of Australia, together with our representative from Germany and one from America. The emphasis for “Coal 2003” is on the longwall environment which is increasingly important for the long term success of the Australian Coal Industry.

It is my pleasure, as Chairman of the Illawarra Branch, to thank all those who organized and participated and to wish you all a successful and rewarding conference.

Ray Tolhurst
Chairman, Illawarra Branch
Australian Institute of Mining and Metallurgy
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KEYNOTE ADDRESS

NEW REGULATION FOR HEALTH, SAFETY AND SUBSIDENCE

Rob Regan

INTRODUCTION

As you may be aware, the Occupational Health and Safety Act 2000 applies to every industry in New South Wales, including the coalmining industry. However, despite this, the Government has determined that in addition, specific coalmine health and safety laws should remain in force in this State.

The Coal Mine Health and Safety Act 2002 (The Act) replaces the 20-year-old Coal Mines Regulation Act 1987 (CMRA) with modern legislation that aims to better protect the health, safety and welfare of people who work in the New South Wales coal industry.

The Act is complementary to the more general Occupational Health and Safety Act 2002 and is to be read as if part of that Act. It provides a framework to manage the particular risks arising from coalmining, and lays the foundation for an integrated approach to mine safety through the development of health and safety management systems, major hazard management plans and emergency systems.

The central features of The Act are:

- application to all places of work within a colliery holding under the Mining Act 1992
- nomination of an operator for any coal operation by a colliery holder
- that the operator must be the employer with day-to-day control of a coal operation, and
- that a coal operation may be an underground mine, an open-cut mine or a coal preparation plant.

A central element of the Act is the requirement that operators develop and implement a comprehensive health and safety management system as a condition for mining to be undertaken.

At the present time a variety of rules, schemes, systems and plans are required under the CMRA to be prepared by a mine manager. The Act consolidates those requirements into a single, integrated and comprehensive health and safety management system.

The various rules, schemes, systems and plans will become important elements of the integrated system. Health and safety management systems will be required to cover such matters as major hazard management plans, the management structure for a coal operation and a contractor management plan.

The systems are comprehensive and cover all those at a coal operation, including employees, visitors and contractors. To maintain existing arrangements, training requirements for the systems will need to be compatible with training schemes required under the Coal Industry Act 2002.

An important part of an operator’s health and safety management system will be a management structure. The management structure must include competent persons to perform key health and safety related functions. The Department of Mineral Resources Safety Operations Unit will monitor the ongoing operation of health and safety management systems. The inspectors will continue to have prohibition and improvement notice powers to ensure that identified safety deficiencies are remedied.

The Act requires that effective emergency provisions are also developed and maintained at coal operations. An emergency management system must be developed. This system would operate separately from the health and

1 NSW Department of Mineral Resources
safety management systems for two important reasons. First, it reinforces the importance of adequate emergency preparedness. Second, it recognises that in an emergency different means of management, such as the formation of incident control teams and the close engagement of external emergency services may be necessary. As with the health and safety management system, an emergency system will cover employees, visitors and contractors at a coal operation.

The Act retains important provisions of the CMRA which are intended to protect the community from potential health and safety impacts of coalmining or to protect the safety of people in adjoining mines. These include ability for the Minister to require:

- leaving barriers or protective pillars in mines,
- closing shafts or outlets in abandoned mines,
- control of emplacement areas, and
- permits for former mines to be used for tourist or educational activities.

To ensure appropriate compliance and enforcement of the new laws, a range of offences, in addition to those contained in the Occupational Health and Safety Act, are included in The Act. Penalties for offences in The Act are at a level commensurate with similar offences under the Occupational Health and Safety Act 2000. When enacted the legislation will be enforced in mines by inspectors and mine safety officers with powers under the Occupational Health and Safety Act.

Another important feature of The Act is the Coal Competence Board, which will replace the Coal Mining Qualifications Board. The Coal Competence Board will oversee the development of competence standards and assessment of people performing particular functions in coal operations. Importantly, the board will be able to continue to arrange for the examination of candidates and the issue of certificates of competence.

Standards of competence for those performing critical health and safety functions in coal operations are essential if risks are to be appropriately identified and managed. Those who work in coalmines need to have the recognised competencies to ensure that they are able to perform their duties without placing themselves and others at risk. The new Act will not commence without regulations being made that recognise these positions and the corresponding competency standards and functions. Those currently in statutory positions will be taken as having the necessary capability to perform the corresponding functions under The Act.

An important part of safety management is to ensure that employees, who often work in challenging conditions, are fit for work and not fatigued. Section 168 of The Act contains important safety provisions regarding powers of work. As part of the modern legislative framework, these provisions are not expressed in the new Act but, rather, will be retained in the regulations. It is important to note that the regulations will be a key component of the safety framework that gives operational effect to important provisions of The Act. To ensure a smooth transition to the new legislation, the regulations will be developed in close consultation with mining company representatives and mineworker representatives.

When necessary, the regulations will be able to make provision for existing arrangements under The Act to be acceptable as fulfilling requirements under the CMRA for a limited period. This will allow existing safety measures to satisfy the relevant requirements of the new legislation while the required work is undertaken to implement new safety standards. The Act provides a basis for a safer coal industry in New South Wales.

**OBJECT OF COAL MINE HEALTH AND SAFETY ACT 2002**

The objectives of this Act are:

- to assist in securing the object of the Occupational Health and Safety Act 2000 in relation to coal operations\(^1\) (including the object of securing and promoting the health, safety and welfare of people at work at coal operations or related places),
- to put in place special provisions necessary for the control of particular risks arising from the mining of or exploration for coal, and

\(^1\) “coal operations” means a place at which coal is mined that is a place of work to which the Coal Mine Health & Safety Act 2002 applies and includes the places that are taken to be part of a coal operation under section 4 of the Act.
• to ensure that effective provisions for emergencies are developed and maintained at coal operations and related places.

OBJECT OF OCCUPATIONAL HEALTH AND SAFETY ACT 2000

The objects of this Act are as follows:

• secure and promote the health, safety and welfare of people at work,
• to protect people at a place of work against risks to health or safety arising out of the activities of persons at work,
• to promote a safe and healthy work environment for people at work that protects them from injury and illness and that is adapted to their physiological and psychological needs,
• to provide for consultation and cooperation between employers and employees in achieving the objects of this Act,
• to ensure that risks to health and safety at a place of work are identified, assessed and eliminated or controlled,
• to develop and promote community awareness of occupational health and safety issues,
• to provide a legislative framework that allows for progressively higher standards of occupational health and safety to take account of changes in technology and work practices, and
• to protect people (whether or not at a place of work) against risks to health and safety arising from the use of plant that affects public safety.

The new legislation is a significant step forward, reinforcing the general duty of care. It provides a powerful opportunity to make mines safer through consolidating risk management with requirements for safe systems. It addresses major issues in the mining sector eg major hazards, emergency preparedness, and contractor management.

It encourages employee involvement in site-based arrangements, further enabling improved safety performance through active consultation for planning and implementation of safety management. The new Act provides for improvements to be made in regulations and advances to be made with guidance material.

STRUCTURE


There are fourteen parts containing 226 sections that provide definitions, application, objects, relationship to the OH&S Act 2000, duties, safety of coal operations, notification of incidents, stop work orders, competence standards, oversight by government officials and workforce representatives, codes of practice, regulations, miscellaneous matters and repeals and amendments.

The new Act focuses clearly on health and safety. In doing so, there are a number of changes including notification of certain "high risk" activities (to be prescribed by regulation) and the removal of requirements to deal with environmental, heritage, cultural and land use issues traditionally administered through section 138 of the CMRA. In regard to approval to extract coal by other than bord and pillar methods, the Act focuses on safety aspects, and other issues will be dealt with under the Mining Act 1992 through lease conditions and the requirement for Subsidence Management Plans.

SUBSIDENCE MANAGEMENT PLANS

The development of the process for Subsidence Management Plans (SMP) had its trigger in the Gretley Inquiry where it was recommended that the process for granting approvals under section CMRA be reviewed.

The Healthy Rivers Commission also presented a report describing ways to protect the environmental values of river systems to the Government.
In response to the Healthy Rivers Commission Report and as the CMRA was also under review, it was decided to separate the approval based on safety considerations from those based on environmental, cultural, heritage and land use.

Section 138 CMRA 1982 was preceded by section 53BA CMRA 1912, which was added in 1964, in order to control resource recovery of private coal. Prior to this mining methods were implemented on a no objection basis.

This reason became redundant when the Coal Acquisition Act 1981 was introduced.

As community expectations changed over time, the power has been used to control safety, environmental, cultural, heritage and land use conflict issues.

APPLICATION OF THE SMP PROCESS

The new process will require the preparation of a SMP and its approval by the Department of Mineral Resources. An approved SMP will be required wherever underground mining will potentially lead to subsidence.

The SMP process will be applied to all underground coal mines. In the case of new coal mines, the key approval remains the development consent and subsidence impacts will primarily be considered as part of the consent process. Subsidence and its impacts must be addressed within the necessary environmental impact statement. The Department will seek to ensure the full integration of conditions imposed under SMP approvals with those imposed under development consents and other approvals. Environmental impact assessment undertaken in association with development applications or other approvals will be taken into account in the SMP assessment process. The Department will seek to avoid duplication in assessment processes and approval conditions.

SUBSIDENCE MANAGEMENT PLANS

The draft SMP accompanying application for approval must include:

- full assessment of the potential environmental, land use and other impacts of that subsidence including significant ecological values, major surface infrastructure, known proposed surface developments, surface features of community significance;
- description of previous subsidence projections and impact assessment associated with any previous development application;
- proposals to minimise impacts of surface subsidence, particularly in areas of environmental, heritage or archaeological sensitivity or important built surface features;
- proposals for ground and surface water management;
- proposals for any necessary rehabilitation of subsidence impacts;
- details of any proposed Community Consultation Process.

Applicants are encouraged to submit applications for SMP approval in respect of complete longwall domains. Applications will also be required if significant variations are proposed to previous subsidence predictions, subsidence impact predictions, or subsidence management strategies and techniques. Extension or variation of the SMP approval may involve further assessment and altered conditions.

When approved, the SMP will form part of the Mining Operations Plan required under the mining lease, and therefore be subject to the requirement for lodgement and review of an Annual Environmental Management Report.

Departmental Approvals Process

The SMP will be subject to the approval of the Director-General of the Department of Mineral Resources. The draft SMP will be assessed by a Departmental SMP Review Committee comprising the Assistant Director Environment (Chair), Chief Inspector of Coal Mines, the Principal Subsidence Engineer, Manager Policy and Legislative Review, and Chief Geologist Coal and Petroleum. The SMP approval process will address development of conditions for the Director-General’s approval.
Interagency Participation in SMP Approvals

An Interagency SMP Review Committee will be chaired by the Department’s Assistant Director Environment and will include representatives from all agencies with significant interests affected by the proposed SMP. The trigger being significance to the agency concerned of the features or values subject to potential impact.

Submission of draft SMPs and Applications for Approval

During development of a draft SMP, applicants must engage in a process of community consultation, and are encouraged to apply the Guidelines for Best Practice Community Consultation in the NSW Mining and Extractive Industries, developed by the NSW Minerals Council.

In association with submitting a draft SMP and application for approval, applicants must advertise in a local and a State newspaper their submission of an application for an SMP approval.

The applicant may include proposals for a Subsidence Community Consultation Process (SCCP) within their draft SMP. An SCCP is a means by which the affected community is advised regarding the terms of an SMP approval, the proposed timetable for approved mining activities, the expected impacts of subsidence, proposed remediation and rehabilitation, and other information regarding the mining process that may be of interest to the community. The community will be given access to the final terms of the SMP approval.

Ongoing Community Monitoring

The SCCP may make provision for the involvement of the community in the ongoing monitoring of mining and mining-related impacts.

Subsidence Monitoring and Reporting Program

The Department may require that the titleholder appoint an Expert Review Panel or obtain an independent environmental audit of subsidence management and subsidence impacts.

The Department’s SMP approval process will include development of advice on any additional security deposit considered necessary.

ENFORCEMENT OF SMP APPROVALS

All the enforcement provisions of the Mining Act 1992 for adherence to conditions of title will apply in the case of SMP approvals. Additional environmental management conditions can also be attached to the mining lease under section 239(2). Applicants should refer to the revised Guidance Notes issued when the process is introduced, in preparing their draft SMP and application for approval.

CONCLUSION

The commencement of the Coal Mine Health and Safety Act 2002 and the Subsidence Management Plan process will introduce significant changes for the NSW coal industry.

Our commitment is to provide stakeholders with sufficient information, education and counsel to ensure a positive transition for the benefit of all stakeholders.
KEYNOTE ADDRESS

HAZARD MANAGEMENT IN LONGWALL INSTALLATIONS

Brian Lyne ¹

INTRODUCTION

From its beginnings in the mid 1960’s, longwall mining has become the mainstay of the current underground coal mining industry in Australia. In addition and contrary to the variances in legislation applying to mines operating in Queensland and New South Wales, the underground coal reserves in both states exhibit the same hazards.

Words in mining legislation may differ from state to state, however the intent has a common theme - management of the safety and health hazards in the workplace.

All longwall mines share three basic features, they all involve the employment of people, they use similar types of machinery and extract coal from seams, which were deposited many millions of years ago. The purpose of this presentation is to briefly review some of the background matters surrounding the current mines and then identify a number of the hazards associated with each of these features. Finally, some comments are offered for the purpose of identifying some areas of opportunity to eliminate or better control hazards in the future.

Longwall mining should be the safest method of mining due to systematic process with high levels of engineering. The fact that injuries continue to occur is evidence that there is room to improve the level of risk control applied at a mine.

QUEENSLAND UNDERGROUND COAL MINES

BACKGROUND

The first longwall in Queensland started at German Creek Central mine in 1986. Since then more mines have opened up in the Bowen Basin in order to extract the rich coking coals.

Queensland currently has twelve underground coal mines, ten of which operate longwall machinery, one is being developed to operate longwall machinery and the other was a longwall mine but is now only using continuous mining machinery. The longwall mines produce approximately 38 million tonnes of saleable coal, most of which is for the lucrative coking coal export market.

COAL RESOURCE

Generally the geological conditions are favourable to high extraction rates with thick seams at gradients generally below one in ten. Seam thickness varies from 1.8 metres up to 7 metres. The working heights varying between 1.8 metres and 4.8 metres.

The presence of faults, dykes, and seam rolls is not uncommon with mines in New South Wales although the presence of low angle faulting and shear zones appear to be more prevalent in Queensland.

Roof strata generally varies from a strong sandstone to a weak banded mudstone. None of the coal seams mined in Queensland have the strong conglomerate roof strata found in several mines around Newcastle.

Bowen Basin coal seams generally contain high levels of seam gas, the predominant gas being methane. It is not uncommon for mines to report figures of in excess of ten cubic metres of gas per tonne of coal. Methane gas outbursts have occurred at several mines during the past thirty years.

¹ Department of Natural Resources and Mines, Queensland
Carbon Dioxide gas is present in some Queensland mines and high levels of hydrogen sulphide gas have been found in relatively isolated areas of a few mines.

Several coal seams are prone to spontaneous combustion and have been identified as the cause of most of the notable disasters in the recent history of underground mines in Queensland.

**ENVIRONMENT**

Heat stress is a matter of concern at several large underground mines. With a high ambient temperature in a tropical climate, the additional heat from large diesel engines and electric motors combine to test the limits of normal physical endurance.

Very few of the current mines in Queensland have significant problems with surface subsidence, and none have any residential areas to consider with. Subsidence of broad acre farms, country roads and electrical transmission towers constitute the main area of concern.

Several mines have extensive gas draining installations all of which vent the methane to the atmosphere with the exception of one mine which flares the gas. Gas drainage is done by either inseam drilling techniques or surface boreholes and goaf drainage from the surface.

**WORKFORCE**

The twelve operating underground coal mines in Queensland employ around 2500 mine workers plus contractors.

Most mines are located in the order of three hundred kilometres distance from the coastal areas. In the early stages of development in the Bowen Basin mines, small mining towns were developed to domicile the workforce and their families. Since then attitudes of society have changed and an increasing proportion of the workforce choose to locate their families in the coastal towns of Mackay and Rockhampton (Yeppoon). The mineworker then has to travel in order to leave the family in a location where there is access to higher education and more extensive medical services.

With both the direct and indirect employment opportunities in the mining towns being comparatively limited, the incentive to live on the coast is further enhanced.

Upon retirement, very few mineworkers choose to stay in the mining towns resulting in the community having very few senior members. As a result, a mine also looses much of its corporate memory resulting in many of the lessons of the past being re-learnt by adverse experiences.

Recognised training organisations are a major training resource used on the Queensland mines due to the distance that regional higher education centres are from the mining townships. To date the achievement of a consistent and satisfactory level of training and assessment standards has proven elusive.

**HAZARDS IDENTIFIED AT MINES**

All mines in Queensland are required to develop and implement a mine safety management system that addresses each of the safety and health hazards at the mine. Hazards with the potential to cause multiple fatalities are required to have Principle Hazard Management Plans.

The following hazards have been identified during either investigations of incidents or the conduct of mine emergency exercises:
Frictional ignition

Several events have been recorded during the past ten years. The events have occurred during both bi-directional and uni-directional operation of the shearer. Ignition is associated with sandstone roof or floor strata with a high quartz content, metamorphic intrusions with a high quartz content or hard pyritic nodules. Cutter pick management on the shearer drums and back flushing sprays have reduced the frequency of incidents but have not been totally successful. One mine is currently using the latest design in ventilated shearer drums in an effort to reduce the risk of ignition to as low as reasonably possible levels.

Strata failure

Weak roof strata, high coal ribs, joint planes parallel to the roadway, low angle shear zones and high and low stress areas continue to present uncontrolled risks to mine workers.

Injuries have ranged from fatalities to loss of movement in the limbs. Rib failure is the most common of the strata failure events and is often associated with less strata movement noise than roof movement events. Hydraulic powered face sprags fitted to the roof supports significantly reduce the risk level to persons conducting repairs and maintenance on the shearer, however their design is unable to prevent all uncontrolled coal falls along the working face.

Meshing of ribs in access roadways provides a high degree of security, however to date it has not been widely done on a consistent basis similar to roof strata support.

There appears to be a perception at all levels in the workforce that the risk from rib falls is lower than for roof falls and therefore the additional and immediate support is not warranted. This perception must change.

Gas outburst

Gas outbursts are not a common event in Queensland mines with only one being reported on a longwall face and one in a development heading during recent years.

In that instance, gas drainage had been conducted in the development heading but was ineffective due to the surrounding disturbed strata blocking the drainage holes.

With mines continuing to extract the deeper coal reserves, the risk of outbursts is increasing.

Spontaneous combustion

Spontaneous combustion is a hazard that is potentially present in most mines. Whilst statistically it might occur on an infrequent basis, it has been the cause of several mine disasters and the loss of a full longwall face at one of the mines. Several other mines have recorded high levels of carbon monoxide gas in the longwall goaf area, which has resulted in the withdrawal of the workforce from the mine.

All mines conduct the standard R70 test, however experience at mines would suggest that the test should only be used as an indicator. It is imperative that every mine has a calibrated gas monitoring system and surveillance by competent persons in areas where evidence of spontaneous combustion may be found. Different seams have different characteristics that need to be identified. Text book indicators such as carbon monoxide make in litres per minute, are very dependent upon the seam characteristics, method of working and method of ventilation.

Spontaneous combustion is recorded as the cause of the disasters at Box Flat, Kianga and Moura No 2 which occurred in 1994, the latter incident had a profound effect on mining legislation in Queensland. It is of value to review the gas monitoring results of the 512 panel in Moura No 2 covering from when the panel was sealed until after the second explosion. (See Graph No1 and Graph No 2 in Figure 1).

Mining Engineering students will find the gas monitoring graphs of particular interest. These graphs were taken from the Simtars report into the 1994 Moura No 2 incident and is arguably the best gas monitoring record of any coal mine explosion in the history of coal mining in Australia. The graph demonstrates that technology can be an important aid to controlling hazards but cannot guarantee that there is no residual risk of significant proportions.
As can be seen from both graphs that the atmosphere, after the first explosion, has high levels of methane gas and low oxygen. The methane is believed to have come from old sealed areas where the seals had been demolished. The high levels of methane prevented mines rescue attempts and also played a major part in dropping the oxygen levels to levels where persons without self contained breathing apparatus could not escape. This data was vital in the developing the logic behind the new regulations for the strength of seals and the need for oxygen self-rescuers.

**Graph No 1**

![Graph No 1 - Graphs of gas monitoring at Moura No 2](image)

**Graph No 2**

![Graph No 2](image)

**FIG. 1 - Graphs of gas monitoring at Moura No 2**
The panel was being monitored using tube bundle gas monitoring equipment, which was calibrated during the 24-hour period after sealing. The trend graph clearly shows that the oxygen levels were decreasing at a consistent rate whilst at the same time methane and carbon monoxide were increasing at a consistent steady rate. The graph demonstrates that monitoring alone will not necessarily prove the existence of a spontaneous heating.

**Stored energy**

The risk of stored energy in the water emulsion hydraulics system is well known. Fortunately the occurrences of fine oil sprays injecting into mineworkers is rare but not unknown. Penetration of hydraulic fluids into the human body can have serious health consequences. It should not happen however controls remain a matter of training, close observation and maintenance management.

**Respirable dust**

Respirable dust was recognised as a health hazard by Agricola circa 1550. It took 400 years to develop systems that were effective in measuring and controlling the risk of pneumoconiosis in coal mines. Regulation controls are in place to limit the exposure of persons to less than 3 milligrams per cubic metre. Routine dust monitoring has identified that modern high output longwalls are finding it increasingly difficult to meet the regulatory standard, particularly when production levels in excess of 4000 tonnes per shift are mined. Reliance on personal protective equipment is not a satisfactory long-term solution and the matter is currently being targeted by the inspectorate.

**Roadway dust**

Modern shearsers do create large volumes of respirable and roadway dust. Current regulations require the return airways within 200 metres of a longwall face to be maintained at 85% incombustible dust levels. Achieving this level is very challenging for mine operators. Current practice still revolves around the use of stonedust during the production cycle. To date no longwall installations have utilised large capacity dust-scrubbing fans in the return airway or alternative dust inhibitors.

**High voltage electrical power**

The energy requirements to operate a modern longwall face continue to increase with substations now being installed up to 6.5 MVA. In order to transmit the required energy levels to the shearer and maintain a cable with manageable dimensions, the voltages have now risen to 3.3Kv. Issues of concern include:

- Fault levels and control apparatus have been found to exceed the available technology giving rise to open sparking events.
- Electrical maintenance is critical to the ongoing safety and the ability to service the control apparatus requires the employment of specially trained mine electrical personnel.
- Transient over voltages that cause flashovers in switchgear.
- Uncontrolled movement of remote controlled equipment.
- Capacity of flameproof enclosures to contain high power electrical arc faults.

**Heat stress**

Mining in areas where the ambient temperature of the surface contributes to the high underground temperatures means that the addition of diesel vehicles and wet roadways pushes the effective temperature to the limits allowed by regulation. Predictably, summer is the most problematic period.

One central Queensland coal mine has adopted metalliferous mining technology and has installed two large air-conditioning plants to cool the intake airways. The installations have a combined cooling capacity of 3.6 MW resulting in the underground temperature falling by and average of 3°C for an airflow of 196 cubic metres per second.

Air conditioning plants commonly use water evaporation to assist in the cooling process. Legionnaire’s Disease has been identified as a possible hazard for staff working around the air-conditioning plant.

Where heat stress is a possible risk, the medical support systems at the mine need to be able to cope with heat illness. This includes training of first aid personnel at the mine and awareness of heat illness for mineworkers who need to be able to self-monitor for symptoms.
Musculoskeletal injury

Musculoskeletal injuries are the most common injury occurring in mines. Longwall installations have their own unique risks as well as those commonly found in mines such as wet boggy and uneven floors. There is a need for improved ergonomic design. Injuries have been recorded as a result of:

- heavy spare parts being manoeuvred in a confined spaces.
- walking along the face line operating machinery where the working height is limited by the chocks.
  Head and neck injuries are a high risk.

One of the newest longwall installations in Queensland extracts a 1.8 metre thick seam. The large chocks make access very difficult for persons who are unaccustomed to the working conditions. The mine has installed a special chair to assist workers stretch their back for some relief during the shift. Fitness conditioning of plant operators is a matter that requires careful monitoring.

The long term health effects of working in this type of environment for twelve-hour shifts is not known and will be a matter for future research.

Emergency rescue and escape

Queensland conducts an annual “Level 1” emergency exercise in which the mine is tested in its ability to respond to an emergency. The exercises are very realistic and tailored to possible events at the mine. Hazards identified by these exercises include:

- difficulty in extracting seriously injured persons along the chock line
- difficulty in accessing the return airway due to the end chocks being designed to eliminate goaf material rilling into the walkway area
- extreme difficulty in recovering injured persons from a longwall face should the intake airway be blocked by an impassable fall of roof and the only access is the return airway. Return airways are often several kilometres in length, a distance that a rescue team could not possibly carry an injured person
- roof falls in the intake airway have the potential to reduce airflow, increase the temperature and goaf gas contaminants in the airstream

Punch mining has potential to reduce some of the hazards and is being strongly considered a several sites. One mine, which operated a punch mine from an old open cut excavation recently advanced the face close to the high wall of the open cut, withdrew the underground workforce and removed the chocks by open cut methods.

Atmospheric gas contaminants

Contamination of the atmosphere in a longwall recovery face using large diesel machinery to move the equipment, has been found to be a problem because high levels of carbon dioxide and nitrous fumes can be present. Auxiliary ventilation and /or the use of large electrically powered prime movers are often used with success.

Noise

Hearing loss is a major compensation cost to the industry. Noise levels on longwalls commonly exceed 85 decibels and require the use of hearing protection.

Inrush

Problems experienced in this area have generally been associated with extracting panels under open cut voids which were filled with water. The events have been avoidable and fortunately no persons have been seriously injured as a result of these events.
FUTURE

Longwall face installations continue to improve in design and productive capacity. Not all faces have each of the hazards identified above, however all working faces have many of the identified hazards present. Most hazards on longwall installations have been identified for at least ten years and are not subject to high order controls and therefore continue to present a risk to the health and safety of operators.

There can be no argument that further research is required to assist mining engineers develop high order controls for the many hazards that exist in a mine environment. The controls will need the assistance of modern technology and infrastructure and should reduce the dependence upon persons and training.

All longwall faces have high levels of automation yet still have operators walking up and down the face with each shear. The High Wall Mining equipment used in open cut mines can be remotely operated without a person going underground. There is no technical reason why shearsers and chocks cannot be reliably operated without the need for the operator to walk with the shearer on every shear.

The levels of dust and noise in the working environment demand that operators wear personal protective equipment whilst ever production activities take place. This is another reason to introduce remote control from a place of safety where noise and dust can be better controlled.

Machines need maintenance on a very regular basis, reliability factors need to be improved to levels where maintenance is only required every several hundreds of hours of operation.

Shearer drums create large amounts of potentially explosive dust. To reduce this there is a potential to use old technologies such as a plough, trepanner or other technology such as hydraulic picks that do not break the coal into such small pieces in the high velocity air currents ventilating the working faces.

Frictional ignitions of methane gas continue to occur. Pre-drainage of the gas has reduced the risk of a major explosion, but it has not eliminated the risk with high levels of methane gas often found in the goaf area. Although no ignitions of goaf gas has occurred in Australian mines, there was an ignition of methane gas in a longwall goaf of a South African coal mine in 1997.

“Necessity is the mother of invention” and what we need is some inventors to think outside the box and systematically develop high order controls for each of the hazards identified. Conferences such as these are the ideal venue to challenge the traditional thinking and improve tomorrow’s mining industry.

It is past time that quantum improvements in our risk management procedures were made and high order controls were available for each and every hazard associated with longwall mining.

I wish you well in that quest and encourage everyone at this conference to share their collective knowledge and experience and work together toward a safe industry where occupational health is not at risk.
LONGWALL “PORE PRESSURE” GAS EMISSION MODEL

David Ashelford 1

ABSTRACT: Extraction of coal by longwall mining methods has improved greatly over recent years with tonnage rates per week now surpassing yearly tonnes of a few years ago. As mines go deeper and tonnage rates increase then gas emission also increases. For the purposes of understanding gas emission and planning strategies short and long term, modelling of gas emission using the “Pore Pressure” longwall gas emission technique, illustrated is an attempt to model this phenomenon.

The input data to the model includes the gas reservoir properties relevant to underground coal seams and extraction of coal using the longwall technique to evaluate the release of gas into the mine workings. Variations in weekly production, face airflow quantities, the efficiency of gas drainage levels and differing face widths can be tested using the model.

Mine planners can use the interactive spreadsheet developed through this model to assess the limits of production, ventilation capacity and gas drainage requirements.

INTRODUCTION

Gas emission in a coal mining environment is a function of the mining process and its interaction with the gas reservoir. Planners are aware of the problem right from the start - how much emission and how to deal with the problem are where a model can assist.

The development of the “Pore Pressure” model took account of many gas reservoir and geological parameters of coal seams and allowed variation of mining operations in arriving at a gas emission value. This can be directly related to roadway gas concentrations in return and bleeder airways and can be adjusted for gas capture in an interactive spreadsheet. The process is quite complex involving an understanding of the changes in “pore pressure” resulting from rock mechanics processes associated with longwall extraction and the changes in gas desorption from those strata where the pore pressure has been sufficiently reduced. The level of uncertainty can be addressed by using a probability modelling approach.

GAS RESERVOIR PROPERTIES

The longwall “Pore Pressure” model draws on the following gas reservoir properties for the determination of gas release.

• Measured gas content (Qm) at reservoir temperature
• Gas desorption rate
• Gas composition
• Gas sorption capacity at reservoir temperature
• Seam thickness and mineral matter above and below the working section (ash/density)
• Pore pressure
• Coal and sandstone porosity

These parameters and how they are measured are described by Williams, Casey and Yurakov (2000).

Gas desorption occurs in those regions above and below the working seam, where the pore pressure is reduced, as a result of mining, to below the gas desorption pressure of the coaly gas sources. Within the relaxation zone, permeability is deemed to be sufficiently high that gas emission is determined only by desorption rate, which is a function of the pressure differential between gas desorption pressure and pore pressure. An active region of gas emission occurs between the front and rear abutments. When the relaxed zone and goaf passes into the rear

1 GeoGAS Systems Pty Ltd
abutment, the gas desorption rate is reduced to simulate the rise in pore pressure, with a resulting large reduction in gas emission. Once the gas is released it is assumed to be available for gas capture by drainage or ventilation.

THE MODEL

The model is derived from approaches used in Europe (Boxho et al. 1980) where the gas sources within an empirically defined envelope defining the “degree of gas emission” are summed to arrive at a specific emission value. In the pore pressure model, the “degree of gas emission” is variable according to whether the gas desorption pressure exceeds the relaxed zone pore pressure. The gas desorption pressure is determined by the interaction of gas content and gas composition with the gas sorption capacity of the coal at that particular pore pressure. The European models also suffer from the simplistic assumption that gas emission is directly proportional to production rate. In the pore pressure model, re-elevating the pore pressure by reconsolidation of the goaf was found to be essential to obtain a model match with measured data. Definition of the pore pressure resulting from mining is obtained as output from the finite difference simulator FLAC (ITASCA, 1995).

The model consists of a three dimensional array of elements, each being assigned values reflecting the state of the gas reservoir at a particular instant and with respect to location to the mining face and rear abutment. When calculation of gas released is summed from these elements a total gas emission from coal and rock strata from the surface to the working seam, and from the working seam to the extent of the stratigraphy in the floor is achieved. This three dimensional matrix is nominally eight blocks wide with as many as 300 vertical blocks and 250 blocks in length totalling up to 600,000 elements.

The size of the each element horizontally is a function of face width and a length based on production rate. Vertically the elements are related to the thickness of geological strata above and below the working section (Figure 1).

FIG. 1 – Simplified Block Arrangement for Longwall Model

The dimensions of elements are calculated in the following way:

- **Width**: The full-face width is divided in half with four equivalent sections that relate to the boundaries indicated in the FLAC model (Figure 2, NB: Only half the face width is indicated.)
- **Length**: Relates to production rate as a time slice per day: eg 5,000 tpd from a 2.5 m extraction height equals 7.1 m/day. (This value varies with production rate.)
- **Depth**: Represents the thickness of each layer from stratigraphy of the borehole, used for the model. (Examples Figure 3). Layers or sections are normally limited to a maximum thickness of five metres, (to assist with calculation accuracy).
As an aid to understanding the modelling output and comparisons between boreholes, all the coal and coaly layers from the entire borehole are summed to give a profile of; this is plotted as Coal Equivalent Thickness, (Figure 4).
For each element, the following is specified:

- Gas content (in m$^3$/t, m$^3$/m$^3$ and m$^3$/m$^2$), described by Williams (1996) for each element is initially assigned from defined gas content data or more often as a relationship with depth from borehole gas content analysis.
- The pressure of the fluid in the pore volume induced by mining (normally from FLAC modelling, Figure 2).
- Desorption pressure is calculated from the gas content using gas sorption isotherms, (Figure 4).
- Gas desorption rate, which is a function of the gas content, and the extent to which the gas desorption pressure exceeds the mining induced pore pressure. Gas is not allowed to desorb below the gas sorption capacity at one atmosphere absolute pressure.
- Reductions in gas desorption rate in response to goaf loading, set at a prescribed distance behind the face position.
- Proximity to the face and working seam.
- Non-coal stratigraphy such as sandstone and conglomerates assume pore spaces and these are assigned proportional gas content.

**FIG. 4 – Coal Equivalent Thickness and Gas Desorption Pressure at Initial Gas Content**

When the model is run, the pressure for each element is recalculated as per the pore pressure induced by mining and if this is below it’s desorption pressure then gas is released. In an example, say the initial desorption pressure was 1,200 kPa $\alpha^2$ 5.9 m$^3$/t then a pressure reduction to 800 kPa would mean a gas release of 1.4 m$^3$/t at a rate defined by the gas content and the pressure difference (Figure 5). Gas release does not occur below the one atmosphere pressure.
FIG. 5 – Sorption Isotherm Indicating Gas Release with Pressure Reduction

A zone of influence can be plotted where the pore pressure is less than the desorption pressure allowing gas release, (Figure 6).

It is this calculated volume of gas release in each element that is then summed for differing production rates to calculate the total gas make. The initial output is identified points that are fitted with a power trend curve as a gas make curve, (Figure 7).
GAS MAKE

Why model Gas Make? Gas make (m$^3$/t) as a function of production when plotted as a consistent relationship indicates similar gas domains. Complete longwalls or large areas of longwalls have been found with consistent gas domains. A relationship has been established between the gas make coefficients linking gas emission and production rate (Williams, Maddocks and Gale (1992). This relationship will be explored later to explain the use of the modelling result.

As a means of comparison three examples of actual curves have been included to show the gas domains that occur while longwall mining.
- Example 1 (Figure 8) is a typical Gas Make curve from a standard longwall operation.
- In Example 2, the longwall has continued with three separate domains indicated.
- The third chart (Example 3) indicates additional gas emission issuing from a dyke crossing diagonally across the face. Note the return to normal gas make levels after the dyke disappeared from the face area.

Example 1
FIG. 8 - Gas Make Curves of Actual Longwall’s

GAS EMISSION MODEL CAPABILITIES

The longwall gas emission model is capable of quantifying the gas make depend on changes in:

- Gas content of the coaly material
- Coal sorption properties
- Porosity and water saturation of the non coaly material
- The thickness and proximity of the gas sources to the working seam
- Longwall face width

Using the interactive spreadsheet demonstrated below changes can also be made to the following

- Mining rate, including production days per week
• Ventilation airflows along the faceline and bleeder airways
• Gas capture and working seam predrainage

The gas make curve from the “Pore Pressure” model alone is only the first stage of modelling. It is the interpretation and information that is obtained from this initial modelling in the form of an interactive spreadsheet that is the real advantage of the model.

An example of an interactive spreadsheet with the gas make curve coefficients entered has variable options in the form of drop down boxes, (Figure 9). In this case the variable options are:

• Daily Production
• Number mining days per week
• Face Airflow
• Bleeder Airflow
• Gas Drainage – percentage and
• Bleeder face separation %

![Example Longwall Emission Model Input/Output](image)

The indicated gas emission results for a daily production of 6,000 tonnes are 1,184 l/s and with 25 % to drainage and 30 % to the bleeder return have the Peak CH₄ percentage in the face return at 1.04%. (NB: Peak Value is the calculated value from modelling times an irregularity coefficient of 1.5)

Altering the daily production to 8,000 tpd changes the Peak Daily CH₄ gas emission to 1,335 l/s and the Peak CH₄ gas percentage in the face return to 1.17%.

**GAS EMISSION MODEL LIMITATIONS**

Only regular gas emission can be modelled. The emission model cannot simulate the propensity for sudden gas releases from floor breaks. The emission model elements are not capable of interaction with each other.

Apart from the effect of goaf reconsolidation, gas desorption rate is the limiting factor in the supply of gas to the system. Included in the goaf reconsolidation effect is the re-establishment of hydraulic head in the floor behind the face.
Gas emission model strengths are its ability to give:

- A reasonably plausible mechanism (compared to traditional European approaches).
- A high level of geological and gas reservoir detail

**CONCLUSIONS**

The modelling scenario offers a tool that can assist mining planners to understand and control longwall gas emission. Its most accurate application is where settings are defined in matching actual emission. In Greenfield sites, where input parameters and model setting can be less reliable, the accuracy of any result is limited by this uncertainty. As experience increases over time, the accuracy of modelling will improve.

**ACKNOWLEDGEMENTS**

I would like to thank all the staff at GeoGAS Systems who have assisted with this modelling program. Special thanks to Ray Williams for his mentoring of the project and also to Eugene Yurakov for his assistance with the gas reservoir calculations.

**REFERENCES**


Williams R.J. 1996. Application of gas content test data to the evaluation of gas emission and hazards during longwall development and extraction. *Symposium on Geology in Longwall Mining* November 1996, pp.35-40

THE USE OF SONIC VELOCITY LOGS TO DEFINE POTENTIAL GOAF DELAMINATION HORIZONS

Barry Ward

INTRODUCTION

This paper presents the findings of part of ACARP Project C9003 (Green & Ward, 2002) which examined the possibility of deriving quantitative guidelines for the use of sonic velocity logs in the identification of potential goaf delamination planes, with a view to improving predictive capability for the delineation of heavy roof conditions.

The downhole sonic velocity log is widely used for the interpretation of overburden strata into geomechanical units and for identifying thick or strong sandstone layers in the main overburden. It can also depict discrete weaker horizons that can act as goaf delamination planes within such layers as high transit time (low velocity) spikes. However, delineation of these planes becomes subjective if the contrast between the peak velocity and the background velocity diminishes.

In terms of predicting goafing behaviour the question then arises as to whether the potential for bed separation can be predicted on the basis of the sonic velocity contrast alone. There were no quantitative guidelines for using the sonic velocity log for this purpose.

The research carried out under C9003 was thus directed at establishing the value of the sonic log for the delineation of potential goaf delamination planes. The proposal was to systematically test sonic log responses for potential separation planes against monitored goafing behaviour from a number of mine sites, with the objective of deriving quantitative guidelines for the identification of goaf delamination planes from the sonic velocity log.

BACKGROUND

It is believed that the first systematic use of the sonic velocity log for the prediction of delamination planes was in the late 1980’s, for defining roof conditions in the 600’s block at Southern Colliery. A typical Southern Colliery 600’s log is shown in Figure 1. The immediate roof was an homogeneous fairly massive strong sandstone up to 9m thick, with an average strength of around 80 MPa. However the sandstone invariably contained one or two persistent thin siltstone partings that acted as separation planes. The term ‘active sandstone’ was coined to refer to the sandstone component directly overlying the seam (Paterson and Ward, 1994). The active sandstone was subsequently defined as ‘the thickness of sandstone up to the first potential delamination plane as indicated by the sonic velocity log’ (Paterson and Ward, 1994).

Extraction in the 600’s block suffered intense but short-lived periodic weighting at around 12m frequency. Observation and monitoring during longwall extraction (Frith and Stuart, 1991; Everett, 1992) indicated that the severity and spacing of the periodic weighting was directly related to the thickness of the active sandstone.

The occurrence of the weak siltstone partings was not necessarily continuous or consistent, and the sonic response varied accordingly. Experience on site led to a minimum peak value of 85 \( \mu \)sec/ft being adopted as the cut-off value for delamination to occur. This is equivalent to a sonic velocity contrast of approximately 10 to 15 \( \mu \)sec/ft against the background level.

The validity of using the sonic velocity log systematically on a quantitative basis for predicting delamination planes was thus established at Southern Colliery with respect to strata forming the immediate roof to a height of some 10m to 12m above the seam.

1 Geotechnical Consulting Services
FIG. 1 - Southern Colliery Active Sandstone

Since then attention has been focussed on massive thick sandstone bodies in the upper overburden, triggered by weighting events at a number of mines, including Southern Colliery (700’s), South Bulga and Crinum, and by the widespread identification of sandstone bodies in the Bowen Basin (Esterle et al., 2001).

The Corvus sandstone at Crinum is a massive sandstone up to 25m thick with an average strength of 35 MPa. It occurs as a channel facies within weaker laminated strata within the Corvus - Tieri seam interval at a height of 25m or more above the mining horizon, and was a focus of attention for a time as a major contributor to roof weighting.

Examination of the sonic velocity logs indicated that the Corvus sandstone invariably contained a number of weaker horizons, which could act as separation planes. The question then was whether the total sandstone thickness or an active sandstone component was applicable in defining areas of potential weighting, and if so, what value of velocity contrast could be used to define the potential goaf parting.

A more significant case for definition was the Aquila seam conditions at the Grasstree Project (Ward, 1998). This is illustrated in Figure 2. The overburden comprises 2.5m to 4.5m of relatively weak, thinly laminated siltstone and mudstone, overlain by approximately 20m of strong massive 80 MPa sandstone. The sandstone contains a weak carbonaceous siltstone or mudstone paring in the middle, which shows up as a weaker horizon on the sonic velocity log. The strength of this paring material appears to vary from 20 to 50 MPa across the minesite but in some places the paring grades into the surrounding rock and is no longer discernible.

There is no doubt that a 30 µsec/ft contrast will act as a delamination plane. In terms of predicting goaf behaviour however, the question is at what point (i.e. what velocity contrast) can the paring be discounted as a potential goaf delamination horizon. This could be somewhat significant in that where the paring is ineffective as a delamination plane, the solid 20m thickness of 80 MPa sandstone could render conventional longwall mining unmanageable.
The objective of the research project was to see if potential goaf delamination planes could be identified from sonic velocity logs according to the differential magnitude of the transit time. The proposal was to compare sonic velocity logs to actual goafing behaviour as monitored from surface extensometer installations and see if any relationship could be established.

Boreholes with both sonic velocity logs and surface extensometer installations were selected from a number of longwall mines for this purpose. It was intended to utilise only historical data in order to undertake an initial assessment of the proposition at minimum cost, rather than attempt a more rigorous and expensive field programme specifically for this purpose. It was recognised that this was less than optimum insomuch that the extensometer anchor locations would not necessarily be positioned at the most appropriate horizons to test the behaviour of specific sonic partings. However it was considered that sufficient information would be forthcoming to permit a judgement to be made on the validity of the proposition if the objectives were not met.
When the project was first proposed only 5 surface extensometer sites were known but since then more sites became available, giving a total of 12 installations from 5 different minesites, as listed in Table 1. South Bulga was on the original list but unfortunately sonic velocity logs were not run in the surface extensometer boreholes.

**TABLE 1  List Of Test Sites**

<table>
<thead>
<tr>
<th>Site</th>
<th>Borehole</th>
<th>Panel</th>
<th>Depth to Roof</th>
<th>Mining Height</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern Colliery</td>
<td>DD403</td>
<td>703</td>
<td>139.15</td>
<td>3.0</td>
<td>installation aborted</td>
</tr>
<tr>
<td></td>
<td>DD411</td>
<td>703</td>
<td>185.76</td>
<td>2.9</td>
<td>installation vandalised</td>
</tr>
<tr>
<td></td>
<td>DD412</td>
<td>703</td>
<td>190.73</td>
<td>2.9</td>
<td></td>
</tr>
<tr>
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<td>3.0</td>
<td></td>
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<tr>
<td></td>
<td>RD3381</td>
<td>704</td>
<td>125.38</td>
<td>3.1</td>
<td></td>
</tr>
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<td>no suitable sonic log</td>
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<tr>
<td></td>
<td>DD433</td>
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<td>181.92</td>
<td>2.9</td>
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</tr>
<tr>
<td></td>
<td>DD416</td>
<td>705</td>
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<td></td>
</tr>
<tr>
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<td>308</td>
<td>326.32</td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>Crinum Mine</td>
<td>5592</td>
<td>LW5</td>
<td>188.52</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5593</td>
<td>LW5</td>
<td>159.92</td>
<td>3.6</td>
<td></td>
</tr>
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<td>101</td>
<td>146.80</td>
<td>4.5</td>
<td></td>
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<tr>
<td></td>
<td>RD256</td>
<td>101</td>
<td>119.40</td>
<td>4.5</td>
<td></td>
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<tr>
<td>Dartbrook Mine</td>
<td>DD149</td>
<td>LW1</td>
<td>217.00</td>
<td>4.0</td>
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</tr>
<tr>
<td></td>
<td>RD468</td>
<td>LW4</td>
<td>286.80</td>
<td>4.0</td>
<td>installation failure</td>
</tr>
</tbody>
</table>

**PROPOSITION**

As described above, it has been established at Southern Colliery that small transit time differentials of 10 to 15 μsec/ft were indicative of separation planes with respect to the immediate roof. This applied to the particular condition of first layer goafing, that is, where the lowest roof layer can become detached and cave into the mined void.

It is highly unlikely that a constant small differential would apply throughout the overburden, as the relative amount of strata separation and subsidence decreases with increasing height above the mined horizon. The expectation was that the further away from the source (height above mining horizon), the greater would be the response required to initiate the same effect. In other words, whereas a 10 μsec/ft contrast could generate strata separation at a height of, say, 10m, it would require a contrast of, say 15 μsec/ft to cause the same effect at a height of 20m.

The proposition was thus made that there would be a quantitative relationship between the magnitude of the transit time contrast required to initiate separation, and the height above the mined horizon. It was proposed to test this proposition by comparing transit time differentials against actual caving examples as monitored by surface extensometers.

If the proposition was correct the results should plot as two populations, with a grouping of separation and non separation values, from which a bounding curve could be defined. This would allow the propensity for strata separation at any given height to be defined by a single variable, namely the transit time contrast.
ANALYSIS OF DATA

Transit Time Contrast

Figure 3 shows an example of a sonic velocity log with extensometer anchor locations marked at the appropriate depths. For each log the sonic response over each anchor interval was examined and potential partings identified. A single parting where appropriate was selected for each interval under the following categories:

1. low strength parting or contrast
2. high strength parting or contrast
3. no partings present

Where several potential partings occurred within the interval, the parting with the largest contrast was selected. This was a simplification in that delamination could occur over several partings within the interval, however it was considered that the largest contrast would see the first reaction to any movement.

The velocity contrast and height above the mining horizon was noted for each selected parting. Low strength (high transit time) partings were the obvious choice but high strength contrasts were also noted since, theoretically at least, they could also act as separation planes under bending stresses.
The magnitude of the differential transit times identified ranged from 7 to 66 µsec/ft for the low strength partings, and from 15 to 28 µsec/ft for the high strength partings. Most of the high strength partings were probably due to sideritic bands; where these were selected a check was made against adjacent borehole logs to ensure they represented persistent layers and not individual lenses.

**Extensometer Response**

Having identified a transit time contrast for each anchor interval, the extensometer response was examined to ascertain if any delamination had taken place over the corresponding interval. Figure 4 shows an example of an extensometer record.

The extensometer records were complex and contained a lot of information, not all of which was real. It was not possible within the scope or budget to analyse the records in detail, hence a simple systematic geophysical first break approach was adopted. Because settlement invariably occurred in several stages with varying amounts of differential movement, the analysis was restricted to picking the first break, that is, the first measurable instance of differential movement between anchors.

Examples of first break picks indicating the onset of movement are shown in Figure 4. In this instance anchors 1 and 2 subsided sequentially with differential movement, whereas anchors 3 and 4 moved simultaneously with zero immediate differential movement. First break picks were categorised as either differential movement or no differential, and matched to the selected transit time response between the same anchors.

![FIG. 4 - Surface Extensometer Example](image)

**Results**

The matched data points for horizons to a height of 100m above the seam are plotted on Figure 5. The data is presented as weak and strong partings that correspond to differential movement (strata separation) and weak partings that showed no differential movement (no separation). Strong partings with no differential movement are not shown.
FIG. 5 - Results of Analysis

It is clear from the distribution of data points that there is no discernible differentiation into two populations of strata separation and no separation, the results for both types being scattered throughout. It is also clear that there is no discernible relationship between transit time contrast, height above mining, and propensity for strata separation. The conceptual picture of two separate populations has not been generated, and the results thus refute the proposition. This means that the sonic velocity alone is not a reliable indicator of goaf delamination planes within the general overburden sequence.

This conclusion is of course based on the data set used in the analysis. Of the 12 sites analysed 5 were start-up installations designed to monitor the first goafing event rather than normal periodic caving (4 at Southern Colliery, 1 at Moranbah North). First goafing covers the transitional caving stage from initial spanning to full caving and is generally different to normal caving where fracturing and subsidence occurs in a cyclic pattern (Gale, 2001). However, exclusion of first goafing data does not improve the validity of interpretation.

One of the main sources of potential error is that the extensometers cannot differentiate lateral movement of strata, as all movement is recorded as vertical displacement, which in the extreme case can lead to some kinematically strange results. The bottom two anchors at Southern Colliery 703 panel, for example, show total displacements greater than the height of mining, and also show intermediate periods of no movement that clearly cannot reflect the fall of strata into the goaf.
Although such idiosyncracies can significantly complicate a full quantitative analysis of strata movement, their potential impact in this study was mitigated by using the first break only as an indicator of movement and ignoring all subsequent variation. Nevertheless there was no way of knowing if some of the small differential anchor movements in the data set could be due to shearing rather than separation. However, the data set had only five separations where the differential movement was less than 25mm, and the exclusion of these made no difference to the distribution of data points.

**DISCUSSION OF RESULTS**

The results clearly indicate that goafing behaviour in general is governed by other factors than the presence of weak horizons, bedding planes or modulus contrast between layers. It is suspected that a combination of factors is involved, including bed thickness, strength homogeneity, and the juxtaposition of beds of contrasting character. In other words, more than one variable needs to be considered in the prediction of goafing behaviour.

This does not mean that the sonic velocity log cannot be used to identify potential separation planes, but that it cannot be used to predict which potential separation planes will influence goafing, at least away from the immediate roof.

Subsidence of strata over a longwall panel has traditionally been grouped into three zones. The zone immediately overlying the seam collapses as broken rock into the mining void and eventually fills up the free space through bulking. The height of this initial caving zone is generally taken as 9 times the mined thickness but can be less in stronger strata. In the middle zone the strata are not broken up but tend to incur physical dislocation with induced fractures or bed separation. The height of this intermediate zone is usually taken as 30 times the mined height as a general approximation. In the upper zone above this the strata tend to subside as an intact block without damage.

The goafing behaviour is obviously different in each of these zones and hence it is not unreasonable to expect that different predictive parameters would apply in each case.

In the intermediate zone, from say 20m to 100m above the seam, the presence of thin sonic partings or bedding planes within a rock layer of constant strength does not appear to influence goafing and the propensity to hang up or span is more likely to be governed by the thickness and tensile strength of specific beds. In respect of the Corvus sandstone example at Crinum Mine it would appear that a sandstone body 25m or more above the mining horizon cannot be reliably sub-divided into an active sandstone component on the basis of weaker partings identified from the sonic velocity log.

Only caving within the immediate roof layer would appear to present any opportunity for analysis within the context of this study; the application of the sonic velocity log for delineating the active sandstone component in the immediate roof zone having been established previously at Southern Colliery by observation and monitoring. Accordingly, the values in the data set pertaining to the immediate roof zone only were plotted. The result is shown in Figure 6, which displays values for picked horizons to a maximum height of 23m. Although there is still some scatter, there is now a discernible separation of the two populations as per the original concept, and a line can be drawn between the two populations, albeit tentatively, from 5 µsec/ft at zero height to 40 µsec/ft at 20m. Interestingly, this line passes through a value of 15 µsec/ft at a height of 6m, which matches the value derived from operational experience at Southern Colliery.

The cut-off height of 23m marks the position where the data show a significant scatter. This height could be taken as representing the average height of the initial caving zone for the data set used, which would be equivalent to 7 times the average mining height of 3.3m.

It is proposed that the line shown on Figure 6 could be used as an interpretation guide for estimating active sandstone, at least for a first approximation. The equation of this line is approximately:

\[
\text{critical transit time contrast (µsec/ft)} = 1.75 \times (\text{height above roof} + 2.85)
\]

The critical transit time contrast represents the minimum value for a weak parting on the sonic log, relative to the background strength of the stratum containing the parting, for which strata separation could be expected. The relationship is only valid for the immediate roof zone to a height of about 20m. Obviously experience and
intuition must be exercised in the interpretation of roof behaviour, as local geological and geotechnical conditions could cause variations in ground reaction.

![Graph showing the relationship between height above roof and transit time contrast.](image)

**FIG. 6 - Immediate Roof Zone**

Application of this relationship to the Grasstree Project example shown in Figure 2 would suggest a minimum contrast of 25 µsec/ft would be needed to cause the lower portion of the Aquila sandstone to become detached and goaf separately from the remainder. As the contrast in the example is only 13 µsec/ft, detachment would not be predicted and the Aquila sandstone would be expected to act monolithically. The active sandstone component would thus be equal to the full sandstone thickness of approximately 16m.

**CONCLUSIONS**

The conclusions reached in this study, based on the dataset established from the 12 sites examined, were as follows:

- The sonic velocity log can be used to predict bed separation in the immediate caving zone to a maximum height of 23m above the mined horizon.
- The relationship between the transit time contrast and the height above mining within the critical immediate caving zone can be tentatively expressed by the equation: \( TT = 1.75 \times (h + 2.85) \)
- This relationship can be used to help interpret the active sandstone component where thick sandstone bodies occur within the immediate caving zone above the mined horizon.
- There is no discernible relationship between transit time contrast and height above mining, with respect to propensity for bed separation in the intermediate and upper caving zones at levels higher than 23m above the mined horizon.
- The transit time contrast alone cannot be used to predict goafing behaviour in the intermediate and upper caving zones.
REFERENCES


TAILGATE SUPPORT DESIGN – AN EMPIRICAL AND ANALYTICAL APPROACH

Greg Tarrant 1

ABSTRACT: There are currently no methods that provide mine operators with reliable tailgate support design. The reliance on experience or at worst, trial and error for tailgate support design is a major contributing factor responsible for longwall downtime and has potentially catastrophic consequences.

An ongoing approach to tailgate support design seeks to better understand tailgate strata mechanics and the interaction between the strata, installed support and longwall powered support.

The approach combining empirical and analytical methods for roadway layout and detailed gateroad support design is an alternative to statistically based assessments... It is also expected to provide greater rigour for pillar design where roadway serviceability is a key determinant.

INTRODUCTION

The consequences of a major gateroad roof fall can be catastrophic and include:

- Abandonment of longwall.
- Inability to recover longwall equipment.
- Disruption to ventilation in the goaf which has the potential to initiate spontaneous combustion.

Whilst major events are uncommon, problematic tailgate behaviour is a common occurrence, frequently leading to longwall downtime of days or even weeks. In many cases the appropriate response cannot be achieved because of difficulties accessing the tailgate to install additional support.

An incident of tailgate instability and longwall downtime leads to a loss of confidence in the support design. This typically results in a legacy of costly overdesign extending into future extraction panels.

The current understanding of tailgate strata mechanics, support behaviour and the interaction of powered supports is not sufficient to provide mine operators with confident in the tailgate support design.

TAILGATE STRATA MECHANICS

Discussion of tailgate behaviour is broken down into the following three broad categories:

- The loading environment.
- Geological controls.
- The interaction with the support elements of primary and secondary tendons, standing support and powered longwall support.

Loading Environment

In a typical two heading gateroad layout for longwall extraction, the tailgate within a multiple extraction panel will experience a wide range of loading conditions over its history, these include:

- Driveage either within a virgin stress field or one that has been modified by previous extraction.
- Driveage under the influence of an adjacent goaf as the maingate travel roadway.
- Driveage under the additional influence of the approaching longwall extraction as a tailgate proper.

1 Strata Control Technology Operations Pty Ltd
The behaviour of the roadway on during development will influence its response to subsequent changes in the loading environment. Clearly if the strata has already softened on driveage, any further stress changes will result in an increase in permanent deformation.

The tailgate support design should take into consideration the style and magnitude of deformation experienced on driveage and the integrity of the primary reinforcement system.

**Stress changes associated with longwall extraction**

Determination of the distribution of stress during longwall extraction is an area of continuing investigation.

**Relative weight distribution between goaf and abutments**

Figure 1 illustrates a general model for the distribution of weight between the goaf and the abutments.

![Diagram](image)

**FIG. 1 - General model for the distribution of weight between the goaf and the abutment.**

Consideration of the surface profile in Figure 1 illustrates a portion where subsidence has not yet occurred and a portion where the subsidence has reached a maximum. The tree marked ‘C’ in Figure 1 is fully supported by the abutments and the tree marked ‘A’ is fully supported by the goaf (in this case a supercritical scenario in subsidence engineering terms).

The proportion of weight distributed between the goaf and abutments between trees A and C is dependent, among other things, on geological controls, caving behaviour and the in-situ stress state. The relative weight distributed between the goaf and abutments in the shared region will vary between mines and should be confirmed through field measurement.

In the absence of field measurement, a conservative approach to pillar load estimation is represented by the straight line ‘AD’ in Figure 1, this line representing the highest conceivable load distributed to the abutments. Where field and analytical data exists, the line ‘AD’ has been found to overestimate the total load distribution by at least 25% and in some cases up to 40%.

The curved line connecting point A with D represents a possible division of relative loading between goaf and abutments that is still consistent with surface subsidence behaviour and would more closely agree with field data.
The shape of the vertical abutment
The theory of elasticity provides exact formulation for the distribution of stress about a circular hole within an isotropic, homogeneous and elastic medium. It is worth noting that the stress distribution for this ideal case is independent of the elastic parameters of the medium and depends only on the geometry of the hole.

Openings other than circular or departure from isotropy or homogeneity preclude the exact calculation of stress about an opening. Numerical techniques (modelling) are required to determine the approximate elastic stress distribution. The introduction of rock softening further modifies the distribution of stress to the extent that a single generic equation for the distribution of stress about a longwall would not be expected to properly reflect the real distribution at every mine.

Figure 1 illustrates the current conceptual model for the distribution of vertical stress about a longwall panel in which rock softening has occurred. The key aspects of the distribution include:

- A ramping up of the vertical stress from near zero at the ribline to a peak value on the boundary of the softening zone.
- A decay of the vertical stress abutment away from the peak value.

Absolute stress measurement and stress change monitoring about longwall extraction are in general agreement with the generic distribution shown in Figure 1 and indicate that:

- The distance of the decay part of the curve extends approximately 0.5 times depth.
- The peak stress is often between 2.5 and 3.5 times virgin.

It is beyond the scope of this paper to discuss the relevance of the stress distribution in terms of pillar design, however in terms of roadway behaviour, the shape of the stress distribution curve is considered to be very important to understand tailgate strata behaviour and will be discussed further.

Horizontal stress changes
The importance of horizontal stress changes about longwall extraction is well recognised and forms a fundamental consideration for longwall layout.

Horizontal stress changes about a longwall are the net result from various competing influences, namely:

- The relief of tectonic horizontal stress towards the extraction.
- Response to vertical stress changes (Poisson effect).
- Gas desorption and fluid pressure changes.

The horizontal stress changes about longwall extraction are superimposed on the stress changes that have occurred on driveage. In this respect the level of roadway softening on driveage and the behaviour of the primary reinforcement system all contribute to the final stress state about the tailgate.

Shear stress on bedding
Figure 2 illustrates an example based on FLAC3D modelling of the distribution of vertical stress in the vicinity of the tailgate/faceline corner.

The high gradients of stress that exist in the vicinity of the tailgate/faceline corner impose high shear stresses on bedding within individual layers and on interfaces between layers.

The slip along bedding and interfaces between layers as a consequence of the high stress gradients is considered to have a major impact on gateroad strata behaviour.
FIG. 2 - Distribution of vertical stress about a longwall tailgate, depth 500m, panel width 140m.

Load distribution
The tailgate in a typical longwall layout is located within a region of high stress gradients, where, large variations in stress occur across and along the gateroad towards the advancing longwall extraction.

In terms of the vertical component, the stress gradient is so large and so influenced by geological controls, that the notion of average stress across a chain pillar becomes ill-defined and of limited value for assessment of gateroad behaviour or support requirement.

In terms of the horizontal component, the stress gradient is the net result of relief towards the approaching opening and change (increase or decrease) as a result of the vertical abutment (Poisson effect). Strata softening processes further redefine the horizontal stress regime.

The high stress gradients impose high shear stresses along bedding within layers and along interfaces between layers.

The extent of stress redistribution about a longwall occurs at the scale of the longwall, not the scale of the roadway.

In order to better control difficult tailgate environments, better definition and understanding of the loading environment from driveage to extraction of the second pass longwalls is required. Improvements in computation that now allow realistic simulation of the longwall softening processes in three dimensions together with targeted field measurements of the stress distribution about longwall extraction are expected to provide advances in these areas.

Geological Controls
The preceding discussion highlighted that post driveage, a tailgate is likely to be influenced by the following general stress changes:

- An elevation in vertical stress during extraction of the adjacent wall, when it acts as an maingate travel ling road, and during the time it is a tailgate proper.
- A competing influence of horizontal stress changes from vertical changes (Poisson effect from increased vertical stress) and horizontal stress relief towards the approaching extraction.
- A potentially high component of shear stress along (horizontal) bedding.
The strata sequence is typically composed of geological units of varying strength and deformability properties, together with discontinuities such as bedding and joints, themselves of varying shear strength and shear stiffness properties.

In order to discuss the relevance of the high stress gradient in relation to strata behaviour, it is necessary to outline some basic mechanics principles. Figure 3 illustrates the relative horizontal movement of layered strata under the influence of a uniform vertical load. The key points of Figure 3 include:

- Softer layers (lower Shear Modulus) move further than stiffer layers, all other things being equal.
- Shear stress is generated at the boundaries of the different layers as a consequence of the different amount of horizontal movement.

![Figure 3 - Relative horizontal movement of layers of varying shear stiffness under uniform load](image)

In a sequence containing coal, mudstone and sandstone for example coal would move further than mudstone which in turn would move further than sandstone.

If the relative movement between the layers imposes sufficient shear stress along the interfaces or along bedding within an individual unit, then irrecoverable slip along the interfaces may occur. Depending on the relative stiffness of the layers and strength of the interfaces, a variety of deformation modes are possible.

Consider the coal/mudstone/sandstone sequence under the influence of a high vertical load. The coal is predisposed to move further than the mudstone so the coal/mudstone interface acts to restrain the coal movement, generating shear stress along the interface.

If the coal/mudstone interface fails, then the coal becomes decoupled from the mudstone and moves freely into the opening. Under these circumstances, the coal rib can override the first roof bolt as shown in Figure 4(b).

Figure 4(c) illustrates an alternative scenario where the coal/mudstone interface has remained intact however the mudstone/sandstone interface has failed. Under these circumstances the mudstone may experience an elevation in horizontal stress from the movement of the coal below and may consequently soften. There is a roof shortening process with similar characteristics to that exhibited by driveage within a high horizontal stress zone which would typically produce guttering. In this environment stress relief towards the goaf may otherwise have been expected.
FIG. 4 - Variation in style of roadway behaviour as a consequence of slip along differing interfaces

The two examples above illustrate possible scenarios within an evenly loaded environment where movement of the various layers towards the roadway is more or less symmetrical about the roadway centreline. Consideration of the stress distribution about a tailgate indicates that the potential direction of shear displacement along interfaces is not symmetrical about the roadway centreline, leading to further possibilities for roof behaviour.
Figure 5 illustrates a general style of roof behaviour observed on a consistent basis in a variety of forms in tailgates and in traveling roads adjacent to the goaf. The general behaviour is consistent with that shown in Figure 4 however the sense and magnitude of shear movement is asymmetrical with respect to the roadway centreline (in response to the asymmetrical stress distribution). The mechanism has been observed to continue to the point where the immediate roof has been pulverised against the block side. On the basis of observed behaviour, installed long tendons in this environment have offered little resistance to the shear behaviour.

![Diagram](sandstone_mudstone_mesh_sheet.png)

**FIG. 5 - General style of observed excessive roof movement towards goaf – displacement controlled**

**Strata behaviour**
Shear displacement along interfaces and within individual geological units is recognised as an important mechanism controlling roadway behaviour generally.

During drivage, it is self evident that the strata softening and stress redistribution processes are related to the scale of roadway. In general the displacements are intimately linked to the stress changes and the system would be considered load controlled, rather than displacement controlled.

Similarly, during longwall extraction, shear displacement experienced by the surrounding strata comes into equilibrium with the longwall stress changes and stress gradient present.

If one now considers the tailgate within the context of stress changes and movement about an approaching longwall, the roadway would experience an environment where the displacements are imposed upon it. In a sense the roadway is a passenger (or is slaved) to the movements associated with the approaching longwall. In this respect, the roadway deformation may become displacement controlled, rather than load controlled.

In a layered medium the magnitude of lateral movements is dependent, among other things, on the location and extent of shear along the interfaces. In a scenario such as that shown in Figure 5, an immediate roof unit has become decoupled from the overlying unit and is effectively being driven towards the longwall block side, slaved to the movement of the surrounding strata.

Observations of roof shortening in roadways (Figure 5 for example) during the approach of extraction (as a tailgate proper) is consistent with the conceptual model that roadway deformation is partly in response to a displacement controlled environment.

To better control the tailgate environment, clearly the location and mechanical properties of interfaces must be determined in order to assess the potential behaviour under partially displacement controlled conditions. A combination of field measurement to characterise the deformation environment and 3D numerical analysis to better understand the potential behaviour, particularly of slip planes, is required.
Support Elements

Significant advances in the product range for standing supports have been made. The common types and their comparative laboratory load/displacement characteristics are shown in Figure 6.

![Figure 6 - Comparative load/displacement of various standing supports](image)

Clearly there is a wide selection from which the mine operator may choose. The range includes the very stiff, strong types that rapidly load but have poor post failure characteristics to very soft types that are slower to load but maintain peak load over a large convergence range.

If the inclusion of long tendons is added to the list of secondary support choices together with the use of combined systems, then it is unlikely that even the total experience base of all mine operators would be sufficient to account for even a small proportion of the total possible designs over the range of geological and loading environments that may be encountered.

A rigorous design approach that can evaluate the choices available without resorting to trial and error is proposed.

TAILGATE SUPPORT DESIGN

Current Approaches

Statistical evaluation of anecdotal data in relation to tailgate serviceability has led to a characterisation of those circumstances under which gateroad serviceability has been unacceptable. The ARMPS (Mark and Chase, 1977) method has been developed for US mines and a modified version, ALTS (Colwell, 1999) has been developed for Australian mines.

Whilst these methods use the current database of experience to assess potential tailgate serviceability, the methods provide little guidance in regard to the actual support design if poor serviceability is identified.
It is not considered appropriate to use a statistically based approach for support design since no insight into the underlying mechanics of the strata behaviour, the driving forces or factors controlling the strata behaviour is provided. The use of subjective experience alone whether characterised within a broader population of experiences or whether restricted to an individual mine, does not allow assessment of future conditions outside that experience base.

NIOSH have conducted full scale testing of a range of standing support products. Barczak (2000) has developed a computer program (STOP) for use in US mines, designed to assist mine operators in the optimisation of standing support.

Barczak (2000) emphasises that the STOP program should not be used to determine the initial design but is intended to be used to evaluate alternative support strategies once an initial support requirement is established. The program includes optimisation on the basis of cost and handling issues.

It is suggested that a support design type be adopted to bridge the gap between those methods that seek to identify potentially adverse tailgate behaviour (ALPS, ALTS and others) and programs such as STOP that seek to optimise the practicalities of the support strategy once support requirements are defined.

**Proposed Approach**

The objective of any support system is to control the deformation environment by way of intervention by artificial means. In this case, the relative horizontal movement of interfaces either between specific geological units or within individual units is a deformation mechanism that requires consideration in the support design.

The conceptual model for tailgate behaviour includes the notion that shear displacements (along layer interfaces or along bedding within individual units) that occur as a consequence of longwall extraction are imposed upon the micro environment of the tailgate. A component of displacement control is introduced into the deformation environment.

The introduction of a component of displacement control has significant consequences for tailgate support design.

In a completely displacement controlled environment, a certain magnitude of the shear displacement along interfaces and roof to floor convergence would occur irrespective of the installed support or reinforcement. The movement would be by definition, irresistible. The support elements would load up in response to the deformation but would not affect the magnitude or sense of displacement that occurred. Under these circumstances, the function of the standing supports or tendons would be to maintain integrity of the fractured rock mass on a local scale throughout the deformation process.

If the deformation environment is only partially displacement controlled, then possible control measures may include:

- Direct reinforcement by installation of solid bars across interfaces (wire tendons would not be expected to provide sufficient shear restraint).
- Generation of confinement across interfaces through the action of loaded standing supports, this may include preloading supports (perhaps using longwall hydraulics).

At this stage further investigations are required to better define whether the tailgate environment is partly or significantly displacement controlled and to therefore better define the strata mechanics and the role of the standing supports and installed bolts and/or tendons.

Current investigations are directed towards:

- Improved understanding of the deformation environment (load versus displacement control).
- Better definition of the stress distribution in the vicinity of the tailgate.
- Measurement of field loading characteristics of standing supports. This includes evaluation of the zone of influence of standing supports and assessment of the most appropriate standing support pattern.
- Measurement of loads developed in primary roof bolts and secondary tendons.
- Improved 3D modelling techniques (adaptive meshing).
- Modification of the stress environment in the vicinity of the tailgate through modified mine design.
It is envisaged that tailgate support design would include:

- Determination of loading environment including consideration of longwall extraction and pillar geometry.
- Characterisation of geological elements, particularly the properties of interfaces between layers and bedding within layers.
- Assessment of potential shear displacement in addition to potential roof to floor convergence.
- Assessment of roof softening and primary bolt loading on drivage.
- Measurement of field loading characteristics of standing supports or long tendons.
- Assessment of powered support loading and longwall extraction method.

It is anticipated that reliable support design will only be achieved through a better understanding of the strata mechanics and measurement of support interaction, rather than statistically based approaches.

Given the complexity of the loading and deformation environment and the array of possible interactions, it is suggested that 3D numerical modelling with verification through field measurement is an appropriate approach that will lead to reliable support design.

**CONCLUSIONS**

There are currently no methods that provide mine operators with reliable tailgate support design. The reliance on experience or at worst, trial and error for tailgate support design is a major contributing factor responsible for longwall downtime in many mines and has potentially catastrophic consequences.

A conceptual model for tailgate strata behaviour is presented which recognises that:

- Stress change about longwall extraction occurs on the scale of the longwall. The gateroads, in particular tailgates, are subjected to a range of displacements that would occur irrespective of the presence of the roadway. This imposes a component of displacement control, in addition to load control, on the roadway behaviour.
- The high stress gradients about longwall extraction impose high shear stresses on bedding within individual geological units and along interfaces between units. Some of the displacement control occurs as shear displacement along these interfaces.

The laboratory load/displacement characteristics of various secondary support types has been established, however, since the deformation mechanics of the tailgate environment are ill-defined, appropriate support selection and design is not rigorous and is reliant on previous experience or trial and error.

The objectives of current investigations are to:

- better define the stress changes in the vicinity of the tailgate corner
- improve the understanding of strata behaviour in the vicinity of the tailgate corner
- determine the field load/displacement characteristics of the various support types
- define the interaction between gateroad behaviour and the longwall powered supports
- further develop 3D numerical modelling techniques to assess support options and to better understand the functions required of the support elements.

**REFERENCES**


Program (STOP). 19th Conference on Ground Control in Mining Morgantown WV. ISBN 0-939084-56-9
THE COAL MINE ROOF RATING IN MINING ENGINEERING PRACTICE

Christopher Mark\(^1\) and Gregory M. Molinda\(^1\)

**ABSTRACT:** The Coal Mine Roof Rating (CMRR) system was developed ten years ago to fill the gap between geologic characterization and engineering design. It combines many years of geologic studies in underground coal mines and worldwide experience with rock mass classification systems. Like other classification systems, the CMRR begins with the premise that the structural competence of mine roof rock is determined primarily by the discontinuities that weaken the rock fabric. Since its introduction, CMRR has been incorporated into many aspects of mine planning, including longwall pillar design, roof support selection, feasibility studies, and extended cut evaluation. It has also become truly international, with involvement in mine designs and funded research projects in South Africa, Canada, and Australia. Most recently, a new streamlined process to determine the CMRR from exploratory drill cores has been developed. Just three types of information are now required:

- Fracture spacing Rock Quality Designation (RQD) from a standard geotechnical drill log
- Uniaxial compressive strength from standard lab tests, geophysical downhole logging, or axial point load tests, and;
- Diametral point load testing.

The CMRR has been implemented in a computer program, which can be obtained from NIOSH free of charge. The program facilitates calculation of the CMRR from either underground or drill core data. Values from many locations can be saved in a single file, and an interface with Autocad allows CMRR contour plots to be integrated into mine planning.

**INTRODUCTION**

Ground falls continue to be the greatest single hazard faced by underground coal miners. One reason is that mines are not built of manmade materials like steel or concrete, but rather of rock, just as nature made it. The structural integrity of the roof of a coal mine is greatly affected by natural weaknesses, including bedding planes, fractures, and small faults. The engineering properties of rock cannot be fully specified in advance, and varies widely from mine to mine and even within individual mines. Moreover, traditional geologic reports contain valuable descriptive information but few quantitative engineering properties. Laboratory tests, on the other hand, are often inadequate because the strength of a small specimen is only indirectly related to the strength of the full-scale rock mass.

Accurate characterization of the strength of the ground is just one problem faced by rock engineers. Another is how to analyze the behavior of the ground. A number of approaches are available, depending on the type of ground:

- For **hard rock**, characterized by high stress and brittle failure, *elastic continuum stress analysis*;
- For **jointed rock**, where most failure is along pre-existing discontinuities, *discrete element and keyblock analyses*, and;
- For **very soft rock**, characterized by low stress and shear failure, *soil mechanics*.

Unfortunately, most coal mine ground control problems fall in between these convenient extremes. As a result, the behavior of a coal mine roof tends to be complex and difficult to predict in advance. If the actual mechanics are to be simulated effectively, highly sophisticated models that include a broad range of potential failure modes are necessary. While such models have been developed (Gale and Tarrant, 1997), they require extensive material properties and validation with field measurements. Rock mass classification schemes were developed to address these concerns. The most widely known systems, including Deere’s RQD, Bieniawski’s RMR, and Barton’s Q, have been used extensively throughout the world. Rock mass classifications have the following attributes:

\(^1\) NIOSH, Pittsburgh Research Laboratory, USA
• Provide a methodology for characterizing rock mass strength using simple measurements,
• Allow geologic information to be converted into quantitative engineering data,
• Make it possible to compare ground control experiences between sites, even when the geologic conditions are very different.

This last point highlights an extremely powerful application of rock mass classification systems, which is their use in empirical design methods. Empirical designs base themselves upon mine experience, on the real-world successes and failures of actual ground control designs. By collecting a large number of these “case histories” into a single data base, and subjecting them to statistical analysis, reliable and robust guidelines for design can be developed. A key advantage of empirical techniques is that it is not necessary to obtain a complete understanding of the mechanics to arrive at a reasonable solution. Rock mass classifications play an essential role in empirical design because they allow the overwhelming variety of geologic variables to be reduced to a single, meaningful, and repeatable parameter.

The Coal Mine Roof Rating (CMRR) was developed nearly ten years ago because none of the existing rock mass classification systems adequately provided for the layered geology and geologic structures typical of coal mine roof (Molinda and Mark, 1994 and 1994b). The CMRR integrates years of research into geologic hazards in coal mining with worldwide experience using rock mass classification systems. It employs the familiar format of Bieniawski’s roof mass rating (RMR), summing the individual ratings to obtain a final CMRR on a zero to 100 scale. It is also designed so that the CMRR/unsupported span/standup time relationship is roughly comparable to one determined for the RMR. To verify the procedure, field data were collected from nearly 100 mines in every major coalfield in the U.S. In recent years, the CMRR has been successfully used to evaluate ground conditions in many coalfields throughout the world.

Two recent developments should facilitate the integration of the CMRR into geologic exploration and mine design:

• The procedures for collecting CMRR data from drill cores have been greatly simplified, and
• A computer program that speeds calculation and interfaces with mine mapping software is now available.

**DETERMINATION OF CMRR UNIT RATINGS**

The CMRR can be determined from underground exposures such as roof falls and overcasts, or from exploratory drill cores. In either case, the main parameters measured are:

• The uniaxial compressive strength (UCS) of the intact rock,
• The intensity (spacing and persistence) of discontinuities such as bedding planes and slickensides,
• The shear strength (cohesion and roughness) of discontinuities, and
• The moisture sensitivity of the rock.

The CMRR is calculated in a two-step process. First, the mine roof is divided into lithologic/structural units, and Unit Ratings are determined for each. Then the CMRR is determined by combining the Unit Ratings and applying appropriate adjustment factors. The second step is the same regardless of whether the Unit Ratings were from data collected underground or from core.

The procedures for gathering data and calculating the CMRR from underground exposures have remained essentially unchanged since they were first proposed in 1994. An underground data sheet is shown in figure 1. Further details on the collection and processing of underground data can also be found in the CMRR program Helpfile.

Procedures to determine Unit Ratings from drill core were originally presented by Mark and Molinda (1996). These have now been streamlined and updated based on recent research. Just three types of information are now required:

• Unconfined Compressive Strength
• Fracture spacing
• Diametral Point Load (an index of bedding plane shear strength)
### CMRR

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**COMMENTS:**

**FIG. 1 - Underground field data sheet.**
Unconfined Compressive Strength (UCS) Rating

The UCS can be determined in a number of ways:

- Standard laboratory testing;
- Estimation from Sonic Velocity ($V_s$) or other geophysical logs, or;
- Estimation from the Point Load Test (PLT) or other index test.

Any of these is acceptable for the CMRR. In the U.S, where laboratory testing is rare and $V_s$/UCS relationships have not been established, the Point Load Test (PLT) is recommended. The PLT has the advantage that numerous tests can be performed for little cost, because the procedures are simple and minimal sample preparation is required. The apparatus is also inexpensive and portable. The International Society for Rock Mechanics (ISRM) (1985) has developed a standard procedures for testing and data reduction. Another advantage of the PLT is that both diametral and axial tests can be performed on core.

The axial PLT (figure 2) is used to measure the UCS. The Point Load Index ($I_{50}$) is converted to UCS by the following equation:

$$\text{UCS} = K \times I_{50}$$

Where $K$ is the Conversion Factor.

\begin{align*}
\text{(A)} & \quad L > 0.5D \\
\text{(B)} & \quad D_e = \text{Equivalent Core} \\
& \quad 0.5W < D < W
\end{align*}

A comprehensive study involving more than 10,000 tests of coal measure rocks from across the U.S. (Rusnak and Mark, 2000) found that $K=21$ worked well for the entire range of rock types and geographic regions in the U.S. The study also found that the variability of the PLT measurements, as measured by the standard deviation, was no greater than for UCS tests.

Figure 3 shows the UCS rating scale used in the CMRR program. The rating rises more rapidly when the rock strength is less than 35 MPa, and reaches its maximum value of 30 for rocks stronger than 150 MPa.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{fig3}
\caption{Relationship between axial PLT and UCS tests for shale (Rusnak and Mark, 1999)}
\end{figure}
Discontinuity Spacing Rating

Most standard geotechnical core logging procedures include some measure of the natural breaks in the core. The two most commonly employed are the fracture spacing and the RQD. Fracture spacing is easily determined by counting the core breaks in a particular unit, and then dividing by the thickness of the unit. The RQD is obtained by dividing combined length of core pieces that are greater than 100 mm in length by the full length of the core run.

Both measures have their advocates in the geotechnical community. Priest and Hudson (1976) suggested that the two can be related by the following formula:

\[ RQD = 100 e^{0.1L} (0.1L+1) \]  

Where \( L \) = number of discontinuities per metre.

As input, the CMRR uses both the RQD and the fracture spacing. When the fracture spacing is greater than about 0.3 m, the RQD is not very sensitive, so the fracture spacing is used directly. At the other extreme, when the core is highly broken or lost, the RQD appears to be the better measure. Either measure may be used in the intermediate range.

The program uses the following equations to calculate the Discontinuity Spacing Rating (DSR) of core from RQD and the fracture spacing. The equations were derived from the original CMRR rating tables.

\[ DSR = 10.5 \ln (RQD) - 11.6 \] or \[ = 5.64 \ln \text{(fracture spacing (mm))} + 5.8 \]  

The minimum value of the DSR is 20, and the maximum is 48 (see Figure 4).

FIG. 4 – CMRR rating scale for axial point load or UCS tests

Diametral PLT Rating

The bedding that is usually present in sedimentary coal measure rocks generally has an important effect on its strength. The problem is particularly acute with soft rocks like shales. Such rocks may be recovered intact, with RQD=100, and may have a respectable UCS, yet their lateral strength may be one-sixth of their axial (Molinda and Mark, 1996). Since the most severe loading applied to coal mine roof is normally lateral, caused by horizontal stress, bedding plane shear strength is a critical parameter.

Unfortunately, bedding plane shear strength is almost never tested directly in the U.S. The diametral PLT is a convenient index test that may be used as a substitute. In a diametral test, the load is applied parallel to bedding (figure 2). Because the precise relationship between bedding plane shear strength and the PLT is not known, and since it seems unlikely that the same \( K \)-factor used to convert the axial test to the UCS would apply, the new CMRR uses the Point Load Index (IS\(_{50}\)) directly. The Diametral PLT rating values were derived from the original CMRR tables and the data presented by Mark and Molinda (1996), and are shown in Figure 5.
Moisture Sensitivity Deduction

Moisture sensitivity can affect roof stability in several ways. The rock itself may be weakened, or may slake or slough. In extreme cases, rock may disintegrate completely and turn to mud when exposed to groundwater. Clay minerals can also expand, causing swelling pressures in the roof.

The CMRR uses an immersion test to determine moisture sensitivity. A rock specimen is placed in a beaker of water, and given a value of 0 (no effect) to 15 (complete specimen breakdown) depending upon the response. Detailed procedures for conducting immersion tests can be found in the CMRR Helpfile. The moisture sensitivity ratings are then determined using Table 1. If immersion test results are not available, moisture sensitivity can sometimes be estimated visually in underground exposures.

<table>
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<tr>
<th>Moisture Sensitivity</th>
<th>Immersion Index</th>
<th>Rating</th>
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<tbody>
<tr>
<td>Not Sensitive</td>
<td>0-1</td>
<td>0</td>
</tr>
<tr>
<td>Slightly Sensitive</td>
<td>2-4</td>
<td>-3</td>
</tr>
<tr>
<td>Moderately Sensitive</td>
<td>5-9</td>
<td>-7</td>
</tr>
<tr>
<td>Severely Sensitive</td>
<td>&gt;9</td>
<td>-15</td>
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Note: Apply to Unit Rating only when unit forms the immediate roof or if water is leaking through the bolted interval.

Usually, some time is required for contact with humid mine air to affect rock strength. In short-term applications, therefore, it may not be appropriate to apply the moisture sensitivity deduction. The CMRR program now reports both the Unit Rating and the CMRR with and without the moisture sensitivity deduction.

Recently, research was conducted to explore the relationship between the Slake Durability Test (SDT) and the immersion test (Mark et al., 2002). The results are shown in Figure 6. The two tests correlate fairly well, particularly for “not sensitive” and “slightly sensitive” rocks.
An important point is that the fracture spacing (or RQD) is actually a measure of the strength of discontinuities as well as their spacing. Weak discontinuities may break apart during drilling, while strong ones might withstand the rigors of the drilling process. Similarly, if the diametral test results show that the rock fabric or laminations are low strength, it would be illogical to give the rock high marks for discontinuity spacing. Therefore, the CMRR defines the Discontinuity Rating as the lower of the Diametral PLT Rating or the Discontinuity Spacing Rating.

The CMRR Unit Rating is simply the Discontinuity Rating plus the UCS Rating, less the Moisture Sensitivity Deduction (when applicable).

**DETERMINATION OF THE CMRR**

If there is only one rock unit in the roof, then the Unit Rating (plus the Groundwater Adjustment)=CMRR. If there are several units, however, the Unit Ratings must be combined before the adjustments are applied. This is done by determining the “thickness-weighted” average of the Unit Ratings. Only those units that are within the bolted interval (up to the height of the bolts) are included in the average.

**Strong Bed Adjustment**

One of the most important concepts in the CMRR is that the strongest bed within the bolted interval often determines the performance of the mine roof. The strong bed adjustment (SBADJ) in the CMRR depends upon:

- The Strong Bed Difference (SBD), which is the difference between the Unit Rating of the strong bed and the thickness-weighted average of all the Unit Ratings within the bolted interval,
- The thickness of the strong bed (THSB) in metres, and,
- The thickness of the weak rock (THWR,) in metres suspended from the strong bed.

In the original CMRR, the SBADJ was determined using a table. For improved accuracy and to facilitate implementation of the table in the computer program, equation (4) was derived using multiple regression:

\[
\text{SBADJ} = [(0.72 \times \text{SBD} \times \text{THSB}) - 2.5] \times [1 - (0.33 \times (\text{THWR} - 0.5))] \tag{4}
\]

The SBADJ ranges from 0 up to 90% of the SBD. Other rules that apply are that the maximum THSB that can be entered into the equation is 1.2 m, and the allowable range of the THWR is 0.5-2.6 m. The THSB must also be at least 0.3 m, because experience has shown that thinner units cannot be counted on to reinforce the roof.

**Other Adjustments**

Other adjustments include:

- **Groundwater:** The maximum deduction for a large inflow of groundwater is 10 points. Also, the rock
must be at least damp for the moisture sensitivity deduction to be activated.

- **Surcharge**: In most cases, the rock above the bolted interval is approximately the same strength or stronger than the rock within. If the upper rock is weaker, however, it can load the roof beam, and a deduction is made.

- **Number of weak contacts**: Roof failure is often associated with major bedding contacts between rock units. The more of these that are present, the weaker the roof. However, strong or gradational contacts are not considered.

### THE CMRR COMPUTER PROGRAM

The CMRR program is designed to facilitate the entry, storage, and processing of field data. Either core or underground data can be entered, and calculations are updated instantly when a change is made. This allows the user to vary parameters, such as the bolt length, to see their effect on the final CMRR.

Figure 7 shows the underground data entry screen. Drop-down menus are used to enter the data for each of the parameters. In the core data screen (Figure 8), the user has the option of entering PLT test data, and having the program automatically determine the mean UCS and diametral IS\(_{50}\). Otherwise, the user can enter the mean strength values directly.

**FIG. 7 – Comparison of the slake durability and immersion tests**

An important feature of the new program is a built-in interface with Autocad. Data from up to 200 locations can be entered and saved in a single file, along with their location coordinates. The program can create a file for export that includes both the calculated CMRR values and the locations. A CMRR layer can then be created in Autocad for use in mine planning.
FIG. 8 – underground data entry screen from the CMRR program

RECENT APPLICATIONS OF THE CMRR

During the past eight years a number of mine planning design tools have been based on the CMRR. The first, and perhaps the best known, was its incorporation into the ALPS pillar design program (Mark et al, 1994). A large database of longwall case histories was collected from throughout the U.S., and subjected to statistical analysis. The results showed that when the roof was strong (CMRR>65), longwall chain pillars with an ALPS SF as low as 0.7 could provide satisfactory tailgate conditions. On the other hand, when the roof was weak (CMRR<45), the ALPS Safety Factor (SF) might need to be as high as 1.3. It is important to note that while the case histories did not really provide information about whether the pillars were truly “stable” or not, the design guidelines have proved remarkably robust and are very widely used in the U.S. In effect, the statistical approach provided a short cut around the extremely complex mechanics of the interactions between abutment loading, pillar behavior, support response, and entry performance.

Longwall Tailgate Design (Australia)

ALPS was the starting point for an Australian Coal Industry Research Programme (ACARP) project whose goal was to develop an Australian chain pillar design methodology (Colwel, Frith and March, 1999). The project aimed to calibrate ALPS for the different geotechnical and mine layouts used in Australia. Ultimately, case history data were collected from 60% of Australian longwall mines.

The study found strong statistical relationships between the CMRR, the tailgate SF, and the installed level of primary support. Design equations were developed that reflected these trends. The final product, called the Analysis of Longwall Tailgate Serviceability (ALTS), was implemented in a computer program and has become widely used in Australia.

Subsequent to the ACARP project, the ALTS case history data base was nearly doubled in size to include virtually all operating Australian longwall mines in year 2000. The final ALTS data base now represents 31 collieries, and extensive statistical analysis resulted in a new design methodology called ALTS II (CGS, 2002). ALTS II can be confidently applied to any Australian longwall where gateroad serviceability is the principle design criterion. It represents a significant leap forward in that chain pillar design and ground support levels (both primary and secondary) can be assessed interactively, rather than independently of one another.

Cut-and-Flit (Extended Cuts)

Cut-and-flit is the standard development method in the U.S. The traditional 6 m cut length was determined by the distance from the cutting head to the operator’s compartment. With the advent of remote control continuous miners, “extended cuts” up to 12 m long have become common. However, many mines with extended cut permits only take them when conditions allow. Where the roof is competent, extended cuts are routine. At the other extreme, when the roof is very poor, miners may not be able to complete a traditional 6 m cut before the roof collapses.
To help predict when conditions might be suitable for extended cuts, a study was conducted at thirty six mines throughout the U.S. The study found that when the CMRR was greater than fifty five, extended cuts were nearly always routine, but when the CMRR was less than thirty seven, they were almost never taken (Mark, 1999). The data also showed that extended cuts were less likely to be feasible as the roof span or the depth of cover increased (Figure 9).

![FIG. 9 – Drill core data screen from the CMRR](image-url)

### Roof Bolt Selection

To help develop scientific guidelines for selecting roof bolt systems, NIOSH conducted a study of roof fall rates at thirty seven U.S. mines (Mark et al., 2001; Molinda, Mark and Dolinar, 2000). The study evaluated five different roof bolt variables, including length, tension, grout length, capacity, and pattern. Roof spans and the CMRR were also measured. Performance was measured in terms of the number of MSHA-reportable roof falls that occurred per 3000 m of drivage.

The study found that the depth of cover (which correlates with stress) and the roof quality (measured by the CMRR) were the most important parameters in determining roof bolting requirements. Intersection span was also critical. The study’s findings led to guidelines that can be used to select appropriate spans, bolt lengths, and bolt capacities based on the CMRR. The results have been implemented into a computer program called Analysis of Roof Bolt Systems (ARBS).

### Longwall Mining through Open Entries and Recovery Rooms

Unusual circumstances may require that a longwall retreat into or through a previously driven room. The operation is usually completed successfully, but there have been a number of spectacular failures. To help determine what factors contribute to such failures, an international data base of 131 case histories was compiled (Oyler et al., 1998).

The study found that the CMRR and the density of standing support were the two most important parameters in predicting severe weighting-type failures. These failures only occurred when the CMRR was less than fifty five, and when the support density was less than 0.5 MPa. When the CMRR was forty or less, all the successful cases employed a standing support density of at least 1.0 MPa. The study was another example of how the empirical method can result in valuable design guidelines even when the mechanics of the situation are not well understood. In fact, in this instance the empirical analysis clearly pointed to a failure mechanism—overburden shearing in weak rock—that had not been previously identified. In several of the case histories, numerical design techniques had been employed which did not take this mechanism into account, and the results were catastrophic.
Roof Fall Evaluations (South Africa)

The CMRR featured prominently in an important research project sponsored by the Safety in Mine Research Advisory Committee (SIMRAC) and other leading industry, labor, and government organizations in South Africa. The goal of the project was to investigate the causes of fatal roof failures in South African coal mines. A team of recognized experts visited a broad spectrum of mines and collected data at 182 roof fall sites. The study found that roof falls were more likely where the roof was less competent in terms of the CMRR. Another finding was that the CMRR correlated well with roadway widths. Based on data collected in 10 Australian, 8 South African, and forty U.S. coal mines (Mark, 1998; Mark, 1999) see Figure 10, the study also concluded “in South African coal mines, less support is used for comparable roof conditions than either the USA or Australia. This supports previous conclusions that in South African coal mines, the density of support needs to be increased” (van der Merwe, 2001).

![FIG.10 – Relationship between cut depth, CMRR, and depth of cover in U.S. mines](image)

Another SIMRAC study found the CMRR easy to use and robust enough to adequately describe the roof conditions at most South African collieries (Butcher, 2001). It took less than four hours for a trained geologist to become competent with the method. The results seemed more reasonable than those obtained from the RMR, which tended to overrate ground conditions by at least one class (twenty points) due to its lack of sensitivity to the characteristics of bedded strata. Some improvements were suggested for the CMRR, including adjustments for joint orientation, blasting, and horizontal stress. A follow-on SIMRAC project is currently underway.

Baseline Comparison of Ground Conditions (Canada)

The underground coal industry of Canada is small and geographically dispersed. To assist the mines in maintaining world-class safety standards, CANMET established the Underground Coal Mine Safety Research Consortium. One of the Consortium’s first projects was aimed at establishing a “best practice” baseline for conducting geological and geomechanical assessments and applying the findings to geotechnical design.

The CMRR was found to be particularly valuable in the assessment (Forgeron, March & Forrester, 2001). It allowed the Canadian underground mines to be compared with each other and with international benchmarks. Based on the CMRR, many ground control safety technologies developed in the U.S. were found to have direct application to the Canadian mines.

Other Applications

- Tailgate support guidelines incorporating the CMRR have been included in the STOP program (Barczak, 2000).
- Input for numerical models have been derived from the CMRR (Karabin and Evanto, 1999).
- Multiple seam mine design guidelines have been developed that incorporate the CMRR (Luo, Hasycocks and Karmis, 1997).
- Hazard analysis and mapping has been based on the CMRR (Wues, DeMario and March, 1996).
CONCLUSIONS

Roof geology is central to almost every aspect of ground control. The CMRR makes it possible to quantify roof geology so that it can be included in mine planning decisions. Worldwide experience has shown that the CMRR is a reliable, meaningful, and repeatable measure of roof quality.

A wide variety of design tools that are based on the CMRR have now been developed. They address a broad range of ground control issues, and rely upon large databases of actual mining case histories. Without the CMRR, it would not have been possible to capture this invaluable experience base.

The new core procedures and computer program further expand the potential of the CMRR. It is now possible to routinely collect CMRR data during geologic exploration or from underground mapping, complete the calculations, and integrate the results into mine mapping software. Foreknowledge of conditions means better mine planning and fewer unexpected hazards underground.

REFERENCES


Mark, C, 1999. Application of the Coal Mine Roof Rating (CMRR) to extended cuts, Mining Engineering, April, pp. 52-56.


ABSTRACT: Gibsons Colliery is mining the Balgownie Seam at less than 10m below the extracted Bulli Seam. The mine enters the Illawarra Escarpment and the depth of cover increases rapidly over the first 500m until it plateaus at approximately 220m. A number of roof falls have developed in intersections – the roadways have mostly been free of falls unless aligned parallel and coincident with roof joints. Underground observations are that the falls are associated with a roof that has no horizontal confining stress – the falls are either joint bounded blocks or failed cantilevers. A roof support strategy based on forming thick cantilevers and if necessary standing support has been developed. A model for how similar stress regimes could develop in multiple seam longwalls has been developed.

INTRODUCTION

History

Thin Seam Mining Pty Ltd began mining in the Balgownie Seam in late 2001. The Gibsons Colliery portal is located at the Russell Vale site of Bellambi West Mine and enters the Illawarra Escarpment immediately adjacent to earlier entrances to the Bulli and Balgownie Seams. The area has been extensively mined, with workings in both the Balgownie Seam and particularly the Bulli Seam. The Bulli Seam workings date from the late 19th Century, with the last mining being pillar extraction in the late 1930’s and early 1940’s.

At Gibsons Colliery the Balgownie Seam is located 6m - 10m below the Bulli Seam. The initial mining was under first workings in the Bulli Seam and mining conditions were excellent. Later, the workings intersected pre-existing roadways (1970s vintage) in the Balgownie Seam (Figure 1) and found a wide range of conditions – either the roadways were open and still supported on wooden props and beams, or there had been major roof falls. The falls were comprised of massive joint-bounded blocks of sandstone. At the same time, new roadways were being driven under the edges of pillar-extraction Bulli Seam goafs and a number of intersection falls developed.

The nature of these falls, the formulation of a geotechnical model to explain their occurrence, and how the lessons from this can be applied to multiple-seam longwalls is discussed.

Mine plan

The Balgownie Seam is approximately 1.2m thick and lies between 6m and 10m below the Bulli Seam. Depth of cover increases rapidly from 0m to 210m as the workings enter the Illawarra Escarpment (Figure 1). The roof of the Balgownie Seam is a thickly bedded sandstone with an estimated average spacing of bedding partings of about 0.75m in the immediate roof; the upper roof is probably more thickly bedded. The thickly bedded nature of the sandstone makes it well suited to extended-cut mining. There are three well developed joint sets in the sandstone – 355°, 100°, 050°. The unconfined compressive strength ranges from 55 MPa to 124 MPa, and averages 88 MPa.

The low seam thickness allows stable pillars to be formed on relatively small pillar centres. The mine layout is based on 5.2m wide roads driven on 20m centres; pillars are 1.4 -1.5m high once the floor is graded to a width/height ratio of 9 to 10 applies.
FIG. 1 - Gibsons Colliery workings in March 2002 (shaded area shows unmined Bulli Seam coal, contours are depth of cover to Balgownie Seam)

In developing the mine plan, consideration was given to the experience in the USA whereby a pillar factor of safety of 2.0-2.2 is recommended for pillars located under goaf/solid transitions. The USA literature also recommends a minimum factor of safety of 2.5 under remnant pillars or stooks in the goaf – this was not adopted as the knowledge of the Bulli Seam does not allow the identification of such remnant pillars.

MINING METHOD

Place-change mining methods are utilized with a Joy continuous miner and a Fletcher roof bolter. Extended cuts of up to 12m are taken.

Roof support design is based on the recognition that extended plunges are possible and that there are no stress change after development – i.e. no subsequent extraction. From a design viewpoint, the demonstrated stability of the unbolted plunges challenges the usual assumptions regarding roof failure mechanisms against which support is specified. The design approach is based on dead-weight suspension of the immediate roof, and this is implemented as four mild-steel bolts, 1.2m long at 1.2m centres. As will be discussed, this support was to be supplemented in areas of seam interaction. The Jennmar Insta’L system has been used. It is important to note that the limited head room means that a header bolt must be used. Mild steel was specified as a pit standard so that if there was a need for longer bolts they could be readily bent to allow insertion. An important feature of the Fletcher Bolter is that it can only install vertical bolts.

ULTRA-CLOSE MINING

Record tracings and the original linen plans for the Bulli Seam are available and these have been shown to be accurate to within about 2-4m. The Bulli Seam mining used Welsh Bords and there were two generations of pillar extraction, one apparently some time soon after mining and a later extraction. It would appear that the extraction is reasonably well recorded in the record tracings in as much as ground conditions in the Balgownie Seam do relate to some degree to the goaf edge shown on the tracings. It would also appear that there are significant stooks or remnant pillars within the areas marked as goaf.
The Balgownie longwalls of the 1970’s were also located under pillar extraction goaf about 1 – 2km to the west of Gibsons Colliery. Some of the early physical models conducted by ACIRL examined the possible conditions under the Bulli pillars (Figure 2). The experiences of some of the miners and the geologists who worked in the Balgownie longwalls were sought. A range of ground conditions were experienced during gateroad development and these were generally in agreement with the physical models. The ability of the operation to control the roof improved as the transition was made from timber to some of the first roof bolts. The physical models did not include joints: joint-control of roof falls was identified by the Balgownie geologists, and this was incorporated in the Gibsons layout whereby the roadways were oriented to bisect the major joint sets.

FIG. 2 - Example of ACIRL physical model of Bulli/Balgownie seam interaction

ROOF FALLS

Minor roof falls are a characteristic of extended-cut mining. Whilst the sandstone roof is thickly to massively bedded, there are cases where more closely spaced bedding partings define beams that cannot span across the 5.2m wide cuts. The few occurrence of this resulted in falls of less than 0.5m in height and the fall debris was readily cleared from the miner.

When mining has been carried out near goaf edges, and particularly at intersections, or when a roadway was driven parallel to and coincident with a joint, larger roof falls have developed. The likely onset of falls associated with multiple parallel joint sets was evident as the cut was taken, so the plunges were reduced in length and standing support installed.

The intersection falls are of particular interest. Joints could be seen to be open as the face was driven but there was no evidence of movement. The roof was supported with bolts, and W straps were introduced to span the joints. The roof drilling would reveal high-strength massive sandstone in the immediate roof. If standing support was not installed, a fall would develop some time over the next five day period.

Typical falls are shown in Plates 1 to 4. They all had some, if not all, of the following characteristics:

- Thick to massive bedded sandstone
- At least one joint plane exposed in the fall, with rock ‘flour’ on the surface
- An arcuate fracture surface reminiscent of feather edging in massive sandstone roofs during pillar extraction.

Occasionally very large falls occurred with joints on all sides.
FAILURE MECHANISM

In developing a failure mechanism, the key observations were the presence of open joints located at any position across the roof – either centreline or towards the ribline, the exposed joint planes in the sides of the falls, the development of feather edges in some cases, and often the presence of water drippers.

It is considered that the falls relate to the mobilization of joint blocks in massive sandstone. The average joint spacing in the massive sandstone roof is probably in the order of 5m and given the statistical distribution of joint spacing, this can result in some occurrences of relatively closed spaced joints. If there is a joint down the centerline of the roadway the roof is stable. If there is a joint down one ribline an unstable cantilever develops (Figure 3a) and if there are 2 joints exposed then a block fall develops (Figure 3b).

![Figure 3a](image-url)  Onset of cantilever failure when one joint exposed along ribline.

![FIG. 3b](image-url)  Joint block failure when two joints exposed in the roof

An analysis of cantilever behaviour using a tensile strength of 2.5 MPa and a factor of safety of 2.0 indicated that beams with a thickness of less than about 1m could not span across a 5m roadway and this increases to 2.5m for a 7m span as is developed over an intersection (Figure 4). These beam thicknesses are very similar to the height of the roof falls that have been observed above extended cuts in the roadways and in the intersections respectively.
A roof support strategy was developed in response to the intersection falls. The original layout of the mine
recognised the importance of aligning the roadways so as not to be parallel to the major joint sets. The ability
of the Fletcher bolter to install only vertical bolts meant that reinforcement of the vertical joints is not possible. The
use of standing support is not compatible with the place-change operation.

The immediate response was to use 4m long cables installed using hand held drill rigs. Whilst this apparently
stabilized the roof the collar assemblies and cable tails proved to be a substantial workplace hazard in the low
seam height.

Finite element analyses were conducted to estimate the bedding parallel shear forces that may develop in
cantilevers that are not thick enough to span across an intersection. An installed capacity of 42 tonnes/metre was
found to be adequate to resist the onset of shear. This was implemented using 2.5m long mechanical anchored
header bolts installed vertically. The support has been successful in controlling cantilever failures.

Control of block falls (Figure 3b) is only currently possible with standing support.

ONSET OF ZERO CONFINEMENT IN THE ROOF

A key requirement for the application of the proposed failure mechanism is a stress field that results in zero
horizontal stress acting across the roof line. The block and cantilever failures will not develop if there are
compressive stress acting across the joints. The elastic solution of stresses about an elliptical hole aligned so that
axes are parallel to the principal stresses can be used to give an indication of the field stresses needed to induce
tensile stresses in the crown of an excavation. If the vertical stress is a principal stress, and the horizontal stress is
equal to K times the vertical, then tensile stresses will be induced when

\[ 1 - K + 2 K H / W = 0, \]

where W is width of roadway and H = height.

For the case of Gibsons Colliery (W=5.2, H=1.5), tensile stresses are induced if K \leq 0.65, which means that the
vertical stress needs to be greater than the horizontal stress.
Of course, in the mine the principal stresses are not aligned parallel to the axes of the opening – nor is a coal mine roadway an ellipse. Finite element modeling has been used to determine the roof stresses induced in the roof of a rectangular excavation in an inclined stress field. The models assumed a major principal stress of 5 MPa imposed on a 5.2m by 1.4m roadway driven in a 1.2m coal seam, and the minor principal stress was varied along with the orientation of the stress field. The roof and floor of the coal seam were modeled with joints, and the roof, coal, and floor material were assumed to be elastic. The two dimensional finite element code (Phase2) was used and a typical output is shown in Figure 5.

**Figure 5 - The zone of negative values of the minor principal stress developed around a rectangular opening exposed to an inclined stress field.**

Figure 6 presents the height of the zone of negative minor principal stress in the roof of the roadways as a function of the ratio of the principal stresses and the alignment with respect to the vertical. There are no tensile zones developed if the major principal stress is aligned flatter than 50° from the vertical, and for steeper orientations the size of the tensile zone increases with increasing values of the stress ratio.

**Figure 6 - Results of finite element analysis of the height of negative minor principal stress as a function of stress ratio and inclination**
Additional finite element modeling was conducted to identify where adverse stress field could develop in a layout similar to that used at Gibsons Colliery. The key requirements were found to be a low insitu horizontal stress field and elevated vertical stresses which can be expected in the vicinity of pillars. The low insitu horizontal stress is considered to be related to the proximity to the escarpment. An interesting result was obtained when the horizontal stress is 4MPa compared to a vertical stress of 5 MPa. Tensile zones develop under the goaf edge and extend up to the Bulli Seam. This is what is observed and also provides an explanation of water in the Balgownie seam workings. It is known that the Bulli workings are not submerged, but it is anticipated that water is present in the destressed floor of the goaf adjacent to the pillars (See Figure 2). When the Balgownie workings are formed, it is suggested that a low permeability connection via joints is made to the Bulli goaf floor.

**FIG. 7 - Development of tensile conditions above a roadway located adjacent to the goaf edge**

**IMPLICATIONS TO MULTIPLE SEAM LONGWALLS**

There is an extensive body of literature on multiple seam mining, particularly in the USA. Hsuing and Peng (1987) provide a useful summary of the mine design issues, although like much of the literature there is little reported on roof support implications. Hsiung and Peng refer to fracture zones (Zone H, Figure 8) that are present below goaf edges. It is not clear how such fracture zones would be developed in areas where there are increases in both vertical and horizontal stresses such that the deviatoric stress is relatively low – an alternative explanation has been developed. It is speculated that the reported fracture zones are areas of elevated high vertical abutment stress such that when a mine opening is made there are induced tensile zones in the roof. In Figure 9, the results of a finite element analysis are presented. In this model, a chain pillar is simulated as a traction applied to a rigid footing with a load equivalent to 20 MPa stress. Field stresses of 7.5 MPa vertical and 10 MPa horizontal were imposed on the model. The model is therefore representative of a standard longwall geometry at about 250 to 300m under typical Australian stress conditions.
Figure 9a shows the distribution of vertical stresses where it can be seen that there is a concentration of stresses at the pillar edges, as is well known. In Figure 9b, there is a similar concentration of horizontal stresses at the pillar edges and zones of elevated stress oriented at about 40° to the horizontal. At an horizon located at ½ pillar width below the pillar, the maximum horizontal stress is in the order of 10 MPa, or an increase of 33%. Figure 9c presents the orientation of the major principal and minor principal stresses, with the black line enclosing the area in which the major stress is aligned at steeper than 45°. The line is also presented in Figure 9d which shows the distribution of K – the ratio of the major to minor principal stress. When Figure 9d is compared to Figure 6, it follows that the contoured areas inside the black line present the area where tensile stress conditions could develop around rectangular roadways with a width to height ratio of 3.7/1. Different zones would apply for different shaped roadways and for different imposed horizontal stresses. Note that there is a good correspondence between the zones in Figure 9d and Zone H in Figure 8.
It is suggested that roof support in roadways in proximity to overlying chain pillars needs to consider both the possibility of the onset of elevated horizontal stresses and also the onset of tensile stresses. The horizontal stress increases are relatively low compared to typical rock strength so the support design can be based on resisting shear along bedding – vertical bolts would be appropriate. The onset of tensile stress conditions in the roof would lead to failure involving shear on vertical joints. Vertical bolts would not be appropriate in this case and a support regime based on ‘slinging’ the roof from ground above the ribs would be preferred as a massive sandstone roof with widely spaced joints as at Gibsons may not be present.

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REFERENCES

**STUDY OF LOAD TRANSFER CAPACITY OF BOLTS USING SHORT ENCAPSULATION PUSH TEST**

Naj Aziz \(^1\) and Benjamin Webb \(^1\)

**ABSTRACT:** A series of laboratory experiments were conducted on a variety of bolt types to examine the load transfer capacities of different profiled bolts in short encapsulation push testing. A 70 mm section of 150 mm long bolt specimen was anchored in a 70 mm long stainless steel tube using full resin encapsulation. Six types of different profiled bolts and two non-profiled bolts were tested. Bolts with higher profile were in general found to have greater shearing resistance and higher stiffness than low profile bolts. Widely spaced profiles allow greater displacement at peak shear strength, and bolts with no profiles produced very little load transfer capability. Rough surfaced plain bolts showed a significant load transfer capability in comparison to a factory supplied smooth surface bolt which supports the belief that rusted bolts have higher load transfer capability than un-rusted bolt surfaces.

**INTRODUCTION**

In the third Australian Coal Operators Conference, Coal 2002 (Aziz, 2002) discussed the load transfer capacity of bolt surface profile under Constant Normal Stiffness conditions (CNS). The main findings from the study were that bolts with deeper rib profiles offered higher shear resistance at low normal stress conditions while bolts with closer rib spacing offered higher shear resistance at high normal shear stress conditions. Also it was found that the peak shear stress occurred at 60% of the profile spacing. In continuation of the work on the subject a number of studies were undertaken to examine the load transfer capacities of different profiled bolts using three different approaches. One such method involves the use of the Short Encapsulation Push Test. Unlike the tests under CNS conditions, the short encapsulation test is carried out under Constant Normal Load conditions (CNL) provided by the walls of the steel cylinder.

Questions are often asked as to why some bolts have higher and wider spaced profiles while others have shallow and narrow spaced profiles and how does each type react in different ground conditions? The answer to this question depends upon the method of testing. The most common methods used, such, as the short encapsulation pull test have no way of identifying scientifically the role of profile configuration on the load transfer characteristics of the bolt. The conventional short encapsulation test tends to suffer from a variety of operational and inherent defects, which make it difficult to produce repeatable results. Also, the short encapsulation pull test is conducted under CNL condition which generally ignore the changing nature of the confining load due to relative resin /bolt surface displacement. The only effective method of characterising the bolt profile influence is to conduct the tests under CNS conditions. Short encapsulation push test can be considered as a suitable method to examine the influence of profile configuration on load transfer capacity as the technique can be used under a controlled environment which can overcome many of the well known problems associated with the conventional short encapsulation pull testing method, even though the method embraces the principle of CNL conditions.

**SHORT ENCAPSULATION PUSH TESTING**

Figure 1 shows the details of the Short Encapsulation Test Cell. The cell is 75 mm long, which is 50% greater than that reported by Fabjanczyk and Tarrant (1992). The longer length cell was selected in order to permit a sufficient number of bolt surface profiles to be encapsulated in the cell. The cell consists of a machined steel cylinder with an internal groove. The groove provides grip for the encapsulation medium and prevents premature failure on the cylinder/resin interface. As opposed to pull testing, push testing involves the pushing the bolt under constant normal load conditions through the hardened resin. With the use of a digital load cell and extensometer, a full load/displacement history could be obtained. A total of 20 cells were prepared for the study.

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\(^1\) University of Wollongong
ROCK BOLT SAMPLE PREPARATION

Six types of profiled bolts and two versions of plain surface bolts were selected for the study. The first four types of the profiled bolts are Australian manufactured and widely used in Australian mines. The other two profiled types included an overseas bolt and a locally developed new bolt, yet to be marketed in Australia. The surface bolts consisted of a factory supplied bolt which was not yet profiled and a profiled bolt whose profiles were machined off in the laboratory. Table 1 shows the details of each tested bolt. For wider application in Australian mining industry, the first four bolts, namely Bolt Types T1 to T4 were called popular bolts, and the rest consisted of two profiled bolts and two plain surface bolts identified as additional bolts. For obvious reasons all the bolt types were given identification designations.

The rock bolt samples were each cut to lengths of 120mm using a mechanised saw. The equal lengths ensured that all the samples of the same type had an equivalent number of profile ribs and that the ends of each sample were square. All bolts were encapsulated into the push test cells using Fosroc PB1 Mix and Pour resin grout. The uniaxial compressive strength and shear strength of the resin used for the tests were in the order of 70 MPa and 16 MPa respectively. The encapsulated samples were allowed to harden for a minimum of seven days before being tested.

The general arrangement for testing is shown in Figure 2. Information on the load/displacement was monitored on a PC, connected to a Load call and an LVDT of the loading system via a data logger.
RESULTS AND DISCUSSION

Load – displacement relationship

Figure 3 shows typical load displacement graphs of testing Type T2 bolts. The figure shows the results of four tests, and demonstrates the repeatability of the tests with a reasonable degree of confidence. Figure 4 shows the combined load displacement graphs of a group of four popular profiled bolts. Clearly, there are differences in the graphs of different bolts and one notable example is that of Bolt Type T3. This bolt had widely spaced profiles, and the peak load occurred at greater displacement than the rest of the bolts. Table 2 shows the details of the test results for the entire profiled and plain surface bolts. These results are the average values for the maximum load, shear strength, and bolt resin interface stiffness values.
<table>
<thead>
<tr>
<th>TYPE</th>
<th>T1</th>
<th>T2</th>
<th>T3</th>
<th>T4</th>
<th>T5</th>
<th>T6</th>
<th>S1 (Rough)</th>
<th>S2 (Smooth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Profile type</td>
<td>T-Bar</td>
<td>T-Bar</td>
<td>T-Bar</td>
<td>J-Bar</td>
<td>Thread Bar</td>
<td>Thread Bar</td>
<td>Filed (T-Bar)</td>
<td>None</td>
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<tr>
<td>Profile centres</td>
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<td>12.00mm</td>
<td>25.00mm</td>
<td>12.00mm</td>
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<td>--</td>
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<tr>
<td>Profile height</td>
<td>1.00mm</td>
<td>1.60mm</td>
<td>0.80mm</td>
<td>1.50mm</td>
<td>1.24mm</td>
<td>1.5mm</td>
<td>&lt;0.1mm</td>
<td>1.6mm</td>
</tr>
<tr>
<td>Profile angle</td>
<td>22.5°</td>
<td>22.5°</td>
<td>22.5°</td>
<td>19°</td>
<td>4.8°</td>
<td>8.8°</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Profile top width</td>
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<td>2.00mm</td>
<td>2.50mm</td>
<td>1.80mm</td>
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<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Profile base width</td>
<td>3.00mm</td>
<td>3.50mm</td>
<td>5.00mm</td>
<td>3.70mm</td>
<td>3.8mm</td>
<td>3.5mm</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>
Table 2 Push test characteristics of different bolts – Average values

<table>
<thead>
<tr>
<th>SAMPLE TYPE</th>
<th>Popular</th>
<th>Additional</th>
<th>Smooth</th>
<th>Rough</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt Type T1</td>
<td>0.70</td>
<td>1.24</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Bolt Type T2</td>
<td>1.40</td>
<td>1.12</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Bolt Type T3</td>
<td>1.20</td>
<td>1.12</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Bolt Type T4</td>
<td>0.70</td>
<td>1.24</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Bolt Type T5</td>
<td>1.24</td>
<td>1.12</td>
<td>--</td>
<td>--</td>
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<tr>
<td>Bolt Type T6</td>
<td>1.24</td>
<td>1.12</td>
<td>--</td>
<td>--</td>
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<tr>
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<td>1.24</td>
<td>1.12</td>
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<td>--</td>
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<tr>
<td>Bolt Type S1</td>
<td>--</td>
<td>--</td>
<td>0.57</td>
<td>1.01</td>
</tr>
<tr>
<td>Bolt Type S2</td>
<td>--</td>
<td>--</td>
<td>0.57</td>
<td>1.01</td>
</tr>
</tbody>
</table>

The peak load – displacement performances of various bolts are presented in Figure 5. The highest average peak load of 132.56 kN was that of Bolt Type T2. This was 23% greater than that achieved by the Bolt Type T4 at 102.09 kN. The difference between these two extreme values is attributed to the bolt profile heights.

Examination of the average displacement results achieved by the bolt samples in Figure 6 showed that Bolt Type T3 achieved the highest displacement of 4.03mm. Bolt Type T2 followed this, with 2.54mm. Bolt Type T4 achieved the lowest average displacement with 2.05mm. Of the additional bolts tested, it was found that the Bolt Type T6 sustained a displacement of 2.37mm at maximum load, while the newly developed Bolt Type T5 achieved a displacement of 2.019mm. Rough surfaced Bolt Type S1 achieved 1.01mm, while smooth surfaced Bolt Type S2 achieved 0.57mm of displacement at maximum load. A comparative study reported by Aziz, Indraratna and Dey (1999) and Aziz (2002) between Bolt Types T1 and T3 and tested under CNS conditions has indicated that Bolt Types T1 and T3 gave similar comparative displacement patterns but at greater displacement ranges. It is thus reasonable to suggest that wider profile bolts can accommodate greater peak load displacement than bolts with closely spaced profiles. This is considered as an advantage for Bolt Type T3 in accommodating more ground displacement without losing its load transfer capability.

Shear Strength Capacity

The average shear strength capacities achieved by each bolt type are represented below in Figure 7. It was found that Bolt Type T2 had the highest shear strength capacity of 25.89 MPa. Bolt Type T1 with an average shear strength of 22.88 MPa was 11.63% less than Bolt Type T2. The lowest shear strength value of the popular bolt type was Bolt Type T4 at 19.88 MPa, which was 23.21% less than the shear strength value of Bolt Type T2.

The rough surfaced plain bolt achieved a shear strength capacity of 22.35 MPa, and the smooth plain surface bolt achieved 7.71 MPa, which was a large drop in the shear strength values with respect to rough surfaced plain bolt. Bolt Type T5 achieved 25.17 MPa, which was fractionally less than Bolt Type T2, while the overseas manufactured Bolt Type T6 with 21.76 MPa achieved a shear strength capacity 15.95% less than Bolt Type T2.
Fig 3 – Load versus displacement values of Bolt T2

FIG. 4 - load displacement profiles of four popular profiled bolts

FIG. 5 - Average peak load of all the bolts
System Stiffness

The system stiffness is the gradient of the maximum load sustained by a bolt to the displacement at the maximum load of a fully encapsulated bolt. Expressed in kN/mm the average system stiffness for each bolt type is shown in Table 2. It is interesting to note that both smooth surfaced bolts were stiffer than the profiled bolts, however this does not mean that the plain surfaced bolts have greater load transfer capacity as the displacement at peak load was very minimal.

LOAD TRANSFER AND PROFILE DESIGN

Bolt Surface / Resin Interaction

Almost all the load transfer capacity between encapsulation resin and the bolt can be accepted as being attributed to the frictional effect. The level of the frictional force is dependent upon the confining pressure. The magnitude of the changes in peak shear strength with respect to applied normal load is shown Figure 8. The graph indicates that there is an insignificant degree of cohesion bonding between the bolt surface and the resin when the vertical load approaches zero. Figure 9 demonstrates the separation of the resin from a bolt when the cast resin was sawed axially and both halves of the resin shell came off clean from the bolt. In summary the load transfer capacity of the resin /bolt interface is a function of the applied normal load alone.
Profile Spacing

Examination of the average bolt profile spacings, outline in Table 1, found that Bolt Type T3 had the greatest profile spacing with 25mm between profile centres. Bolt Type T2 had a profile spacing approximately half that of Bolt Type T3 with 12mm, while both Bolt Type T1 and Bolt Type T4 had spacings of 11mm. The latter product had a design that is called an overlapped design that produced a general reduction in the effective shearing surface of the bolt. Bolt Type T3 design produced a bolt with a reduced circumferential profile length resulting from the absence of a central spine or ‘flash’. As can be seen from Figure 4, it was evident that the displacement required for Bolt Type T3, to achieve maximum load, was approximately 53% greater than the displacement of Bolt Type T1 and Bolt Type T4 whereas Bolt Type T2 had a peak load displacement of approximately 40%. From this it was evident that an increase in profile spacing has resulted in an increase in the displacement at maximum peak load. The increased displacement required to achieve maximum load resulted in a lower system stiffness of the bolt type.

Profile Height

Testing of Bolt Types T3 and T1 were used to examine the effect of profile height on the shear strength capacity across the bolt resin interface. Bolt Types T1 and T3 were of the same ‘T’ bolt design, possessing similar profile spacings, but had different profile heights. As outlined in Table 1 Bolt Type T3 had a profile height of 1.4mm, while T1 had a height of 0.8mm. However, both Bolt Types T3 and T1 achieved shear strength capacities of 25.89 MPa and 22.88 MPa respectively. Bolt Type T2 achieved a greater shear strength capacity compared to Bolt Type T1. These results are reflected in Figure 6, which represents typical load displacement performances of Bolt Type T1 and Bolt Type T2 respectively.

Bolt Surface Condition

The load displacement shown in Figure 5 clearly indicates that the increase in roughness of the plain surface of the bolt has greatly influenced the shear strength capacity of the bolt. The rough finish of the bolt surface allowed additional grip to be provided between the bolt and resin interface and this reinforces the belief that rusted bolts have greater load transfer capability than a clean bolt of the same type.

PRE AND POST FAILURE BEHAVIOUR

Pre and post failure curves obtained for all the profiled bolt types show that, common to all the bolts tested, the average displacement at peak load occurred at approximately 34% of the profile spacing as shown in Figure 4. The peak load displacement of 34% is almost 50% of the values obtained by Aziz (2002), when examining the load transfer of Bolt Types T1 and T3 bolts under Constant Normal Stiffness condition and that clearly demonstrates the influence of test technique on the result outcome.
The post peak load displacement graphs also depicted different picture for Bolt Type T3 in comparison to the rest of profiled bolts. It showed that the post peak load / displacement profile was higher than the other bolts, indicating the ability of the bolt to maintain greater load transfer capability that others.

CONCLUSIONS

Realistically the application of the Short Encapsulation Push Test technique in evaluating load transfer capability of profiled bolts cannot be accepted as a scientifically recognised creditable technique, as the test is carried out under constant normal load conditions, which is not the case. The profiled bolt surfaced are not smooth, and thus the movement of profiles relative to resin surface would inevitably lead to changes in the vertical load. The application of the system on plain surface bolts is however valid. Nevertheless, the Load transfer capacity assessment is, to a certain extent, warranted because the method overcomes many of the problems associated with the conventional pull testing method, including the effect of resin gloving, host material failure and bolt yield. The test cell provided a standardized environment that allowed testing to focus on profile design only. The tests showed that:

- Rib profile height influenced the shear strength capacity of a bolt.
- Peak shear load occurred on all profiled bolts at displacements equivalent to 34% of the rib spacing, which is almost 50% of the values obtained from testing under CNS conditions.
- Load transfer capacity between encapsulation resin and the bolt is due almost entirely to the frictional affect.
- The rough finish of the bolt surface permits additional grip between the bolt and resin interface and this enforces the belief that rusted bolt surfaces have greater load transfer capability than clean surface bolt.

REFERENCES

Aziz, N.I. Indraratna, B and Dey, A, 1999, Laboratory study of shear loading and bolt load transfer mechanisms under constant normal stiffness conditions, in proc. 18 International Conference on Ground Control in Mining, Morgantown, WV, USA, August 3-5, pp 239-247.
Fabjanczyk M W, Tarrant G C, 1992, Load transfer mechanisms in reinforcing tendons, 11th International conference on ground control in Mining, University of Wollongong, pp.212-21
THE CHALLENGE TO IMPROVE THE PREDICTION OF SUBSIDENCE IMPACTS

Chris Harvey

ABSTRACT: Over the last twenty-five years, due largely to direct monitoring and industry funded research the understanding of mining induced subsidence has been greatly enhanced. A high level of correlation between predicted subsidence and actual ground movement has been achieved, especially where the terrain is fairly uniform. The more critical component is determining the nature of impacts specific levels of subsidence will have on surface features, both natural (cliffs, gorges and rivers) and man made structures (roads, pipelines, houses and commercial buildings). With proposed changes to the approval mechanism for longwall mining operations there will be a greater need to improve the accuracy of subsidence predictions and estimating the resulting impacts upon all surface features. This will require further fine-tuning of the mechanisms used to measure and predict subsidence.

INTRODUCTION

Subsidence and resulting impacts are a direct consequence of mining. Changes in longwall equipment and improved ground stability management techniques have allowed longwall block to become longer and wider. This has resulted in changes to the nature of subsidence related impacts. Along with the variations in longwall layouts the prediction of subsidence parameters is now expected to achieve a higher level of accuracy than was the case in the past.

To some extent improvements in subsidence prediction has been driven by a wide range of subsidence monitoring requirements imposed via the current approval process under section 138 of the Coal Mines Regulation Act 1982 (CMRA) (NSW Govt 1982) and the need for subsidence predictions as part of any environmental impact statement for an underground coal mine. There is generally a growing requirement for the wider community to have an understanding of subsidence and how it will impact upon their amenity, residences, public utilities, and natural surface features.

It logically follows that the challenge for all future longwall operations is to be able to predict the levels of subsidence resulting from longwall extraction (primarily its location and magnitude) and the potential impacts of this subsidence upon a wide range of surface features, both natural and man made. The credibility of these predictions are hence linked to the ongoing acceptance by the wider community of these subsidence impacts and the ability of each mining operation to obtain future approvals to either extract pillars and/or use the longwall method of mining.

EXPLAINING SUBSIDENCE

Subsidence predictions have historically been linked with and directly follow from the measurement of surface ground movement as longwall extraction progressed. For this reason they have traditionally had an empirical focus, with the monitoring of results for a wide range of pillar and longwall extraction areas being used to develop indicative subsidence curves. The work of the late Dr Holla has been fundamental in developing both early and current prediction profiles with the various characteristics for each coalfield being taken into consideration. The characteristics of trough subsidence as illustrated by Holla and Barclay (2000) are shown in Figure 1. In more recent times the development of enhanced surveying techniques has provided an improved monitoring and measurement framework to underpin these predictions. Recently the monitoring technique to provide higher accuracy in subsidence prediction has been three-dimensional monitoring so that the full range of movement of a particular point can be plotted over time and in relation to the underlying extraction of coal.

This empirical approach has been responsible for subsidence prediction being viewed more as an art, rather than a science, which resulted in a certain level of suspicion by people outside of the coal mining industry. It follows

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1 NSW Department of Mineral Resources - Wollongong
that any reporting on subsidence and subsidence related issues must have regard for the target audience. The level of detail, the style of reporting and documentation used for a mine manager and other industry related people is significantly different from what should be used in an environmental impact statement and different again from that used in a public newsletter.

The manner by which the predicted level of subsidence is correlated to specific impacts on a surface feature and the way that this is explained to the general public is potentially just as important as predicting the amount of subsidence. Explaining to home owners that the ground beneath their houses will subside by 800 mm to 1 m, but not to worry as it is within the design limits of the building and in any case the Mine Subsidence Board will rectify any damage, can be very difficult. This also applies to a wide range of infrastructure owners, such as councils, water and sewerage authorities, road, rail and owners of large commercial or industrial buildings. However the real challenge is to develop the same level of skill, expertise and understanding in respect of subsidence impacts on natural features.

Knowing the Surface Features

Current requirements for the granting of a lease (i.e. an Environmental Impact Statement (EIS) followed by Development Consent) ensures that at some stage during the life of an underground mine, all the key surface features, land improvements and structures likely to be impacted by mining are identified. For obvious reasons this is considered to be more relevant and highly desirable for longwall extraction, as the direct impacts upon surface features must at some stage be identified as part of the current approval mechanism under Section 138 of the CMRA.

FIG. 1 - Characteristics of trough subsidence (Holla & Barclay, 2000)

Understanding subsidence can be seen as only the first step in a somewhat complicated process of determining subsidence impacts. While predictive models outlining the extent and nature of subsidence can be refined by monitoring, the actual impact of that level of subsidence upon a specific surface feature requires further consideration and understanding by specialists external to the mining and subsidence review process. In this regard a higher level of understanding as to the engineering characteristics and specifications of each surface feature must be understood so that the scale and nature of the impacts can be predicted.

Throughout the life of an underground mine it is not unreasonable for the scale, nature and relative significance of surface features to change. This is true for both man made structures including residential dwellings, public utilities, historical or heritage buildings as well as natural surface features such as cliff lines, streams rivers, swamps, local and or endangered fauna and flora. The rate and degree to which land development for residential, commercial and industrial uses are promoted and undertaken could mean that an entire range of land use issues and or potential constraints can be imposed upon a mining operation some considerable time after the lease was
first granted. In respect of natural surface features it is not uncommon for communities and conservation groups to be more vocal as mining progresses towards a particular feature and the potential mining impacts became visible and known. Examples of this include cliff lines in the Lithgow area and rivers in the Appin/Douglas Park area.

The guidelines developed by the Department of Mineral Resources to support any application under section 138 of the CMRA, do detail the range of surface features that need to be identified and considered in respect of the potential subsidence impacts. Following on from this has been the development of specific management programs to mitigate and where necessary remediate the potential impacts of subsidence on surface features. This is more common for man made structures such as buildings, bridges and pipelines, especially where the engineering characteristics are documented and fully understood. It follows that to obtain optimum understanding of what man made features both existing and proposed are likely to affect a particular mine layout, an informed and mutually supportive relationship with the local planning authorities is important. The aim being to solve as many issues during the planning phase as is possible.

In respect of natural surface features the situation is not as well defined and understood. It is exceedingly difficult and virtually impossible to determine how much ground movement a cliff line or rock formation can withstand before it fails. Similarly the ability of a stream to accommodate local changes in permeability and surface cracking associated with strain induced by valley closure is difficult to quantify. While some generic guiding principals can be developed more detailed understanding can only come about through extensive baseline research, which could be unobtainable on a wider scale. Hence a formal rating system is seen as the only mechanism, which can be used to identify where the baseline research should be undertaken and an acceptance that some features may be damaged. This involves a structured and systematic means of rating a natural surface feature so that the most significant and least significant features are categorised. Following on from this, further work and research can be applied to specific features, which are less significant but may have a local or related significance to other more prominent features. This allows for detailed fine-tuning of a mine plan where extending or reducing a longwall or pillar extraction panel could greatly alter the potential subsidence impact on that particular feature.

This approach has been successfully developed and implemented for cliff line and rock formations in the Western Coalfields. Work by Harvey (1989), Radloff & Mills (2001), and Waddington Kay & Associates (2002), has led to the fine-tuning of mine layouts at a number of collieries. A similar approach could be taken for other natural surface features such as streams.

MATCHING MINE DEVELOPMENT TO PREDICTED SURFACE IMPACTS

The development of a mine has traditionally been dominated or controlled by mining and economic considerations. That is to say key factors such as stress orientation, geological structures, coal quality and ventilation, get prominent consideration in combination with speed of development and a quick return on invested capital. Potential constraints imposed by surface developments, features (both natural and man made) and the possible impacts of subsidence receive little to no consideration, especially for initial mine planning.

The legacy left by earlier decisions and the extraction of coal from specific locations figure largely in the final layout adopted for a mine. The nature of mining means that if a particular layout proves to be less than appropriate the whole process cannot start again with a clean slate. It is not feasible to “put the coal back” and start again. Hence mine development must contend with circumstances as they present themselves, as well as being flexible enough to accommodate as wide a range of options as is possible.

Surface constraints and environmental considerations have been growing in prominence over the last twenty years. This is directly reflected in the range of issues covered in EIS’s for new underground mining operations, the conditions attached to the development consent and the overall cost associated with gaining all necessary approvals. A key component in recent development consent conditions is the requirement for community consultation or liaison committees, as well as undertaking specific documentation and reporting requirements. Also included are the Department of Mineral Resources’ Mining Operations Plan (MOP) and Annual Environmental Management Report (AEMR) requirements, which focus on each mine managing the environmental and rehabilitation aspects of its mining activities, especially mining related subsidence impacts.

A direct consequence of these trends, especially the heightened focus on EIS and supporting documentation, is that a higher level of accuracy and predictability is required for the preferred mine plan. This in effect determines the overall footprint of any proposed mining operation and hence the surface impacts. There is a need to develop
the most suitable mine plan which can then be justified and defended. However if surface impacts associated
with mine subsidence are not accurately determined and receive secondary or superficial consideration, the ability
to defend a particular preferred option in a Commission of Inquiry and to the community in general is greatly
impaired.

To overcome this particular problem, it would appear that the mine planning process must go through a number
of separate iterations, resulting in an optimum plan. The focus of these separate planning steps being:

- Mining and geological considerations; including key components such as stress orientation, geological
  structures, neighbouring mine workings and ventilation;
- Economic considerations; including development/extraction rates, utilisation of existing mine
  developments and optimising the return on capital;
- Minimisation of subsidence impacts; including dams, pipelines, large buildings, public services, bridges,
  future developments, cliff lines, rivers, specific flora and fauna communities.

The resulting mine plan is then a compromise of each separate plan and can reflect a structured ranking and
assessment process, which can in turn be defended at subsequent reviews or inquiries.

It follows that just as certain mining limitations and constraints must be understood, there is an absolute
requirement to define all surface features and the predicted subsidence related impacts that may occur for each
mine plan options. Base line data and improved subsidence predictions become fundamental components in mine
planning and ultimately matching mine development to predicted impacts.

MINE SUBSIDENCE PLANNING

The concept that subsidence is primarily related to second workings and the distinction between 1st and 2nd
workings is fundamental to all industry people. The general community however does not have the same level of
understanding. Similarly the distinction between assessing any 2nd working proposal in respect of mine safety
aspects versus surface impacts is not clearly understood by the wider community. In this regard the current
approval process under section 138 of the CMRA has been criticised at a number of levels, by community groups,
conservation interests and other government agencies.

Following on from this criticism and a number of reviews of the section 138 approval process, changes have been
developed for all underground coal mining operations, which have the potential for surface subsidence. It is
proposed for all potential impacts, other than mine safety to be dealt with via an approval under the Mining Act
1992. This will require the preparation of a Subsidence Management Plan (SMP) and its subsequent approval
wherever underground coal extraction will result in surface subsidence. Consequently an approval under section
138 of the CMRA will not be granted until an approved SMP is in place.

SUBSIDENCE MANAGEMENT PLANS

Mining which results in surface subsidence impacts must only be undertaken in accordance with an approved
SMP. Hence SMP’s are required to be prepared and approved prior to the development of first workings
associated with secondary extraction either as longwall, mini wall or pillar extraction. Also if first workings have
the potential to result in surface subsidence e.g. due to geological conditions such as weak claystone floor, then an
SMP must be prepared and approved prior to the first workings being undertaken.

These SMP’s must include:

- A full description of the area proposed to be impacted by mining activity, including areas of
  environmental, heritage or archaeological sensitivity;
- An outline of existing mine workings within the application area, the proposed development plan and
  a schedule of the proposed extraction for the period to be covered by the SMP;
- Predictions of the expected extent of subsidence for each longwall panel or other sequence of
  extraction;
- A full assessment of the potential environmental, land use and other impacts associated with the
  predicted subsidence;
- An assessment of the economic and social benefits and impacts of the proposed mine development;
• Extracts of relevant conditions of any development consent held, relevant conditions of other licences held and relevant policies of other agencies (including the Mine Subsidence Board and the Dams Safety Committee);
• Description of subsidence projections and impact assessment associated with any previous development applications;
• Proposals to minimize impacts of surface subsidence, particularly in areas of environmental, heritage and archaeological sensitivity or important man-made surface features;
• Proposed risk management plans for all areas of environmental, heritage or archaeological sensitivity or important man-made surface features, addressing both the planning and operational phases of mining. In the case of major surface infrastructure, risk management plans are required to be endorsed by the infrastructure owner prior to the operational phases of mining;
• Proposals for ground and surface water management;
• Proposals for any necessary rehabilitation of subsidence impacts;
• Results of consultation with affected landowners, state and local government agencies and the general community and;
• Details of any proposed Community Consultation Process.

An SMP can be approved to cover up to seven years of projected mining operations. When approved the SMP will form part of the Mining Operations Plan required under the mining lease conditions and therefore be subject to the requirement for lodgement and review of an Annual Environmental Management Report. Annual Environmental Management Reports, including the report into the SMP will be provided to all agencies with an identified interest. Details of the method for preparation of SMP’s are set out in Appendix A.

THE APPROVAL PROCESS

The SMP will be subject to the approval of the Director General of the Department of Mineral Resources. Draft SMP’s and applications should be submitted to the Assistant Director, Environment. The draft SMP’s will be assessed by a Departmental SMP Review Committee comprising the Assistant Director, Environment (chair), Chief Inspector of Coal Mines, the Principal Subsidence Engineer, Manager Policy and Legislative Review and Chief Geologist Coal and Petroleum.

The SMP approval Process will address:

• Development of conditions for the Director General’s approval;
• Advice on any additional security deposit needed to reflect possible subsidence related impacts;
• Consultation with and/or participation by other potentially affected agencies (including Department of Land and Water Conservation, Sydney Catchment Authority, National Parks and Wildlife Service and Dams Safety Committee;
• Consideration of the need to consult outside expertise as appropriate, in considering and preparing conditions for subsidence related environmental impacts;
• Review of the results of previous on-site monitoring during and after extraction;
• Requirements for the title holder to obtain an independent audit or assessment where circumstances warrant and;
• Appropriate amendments to existing SMP’s and their conditions of approval

A flow chart outlining the process for SMP approval is shown in Figure 2.
FIG. 2 - Process for Approval of SMP’s
CONSULTATION AND PARTICIPATION BY THE COMMUNITY

As indicated previously the desire to involve the local community and potentially affected land owners is a fundamental component of the new approval process. This is also reflected with a number of the recent development consents grant for underground coal mining operations in the State.

During the development of the draft SMP each applicant must:

- Advertise in local and state newspapers their intention to submit an SMP and application for approval for the identified area.
- Identify all land holders and local councils directly effected by their proposals
- Consult with all such land holders and councils
- Take into account the views expressed in responses received

Throughout the process of community consultation, applicants for SMP approvals are encouraged to apply the “Guidelines for Best Practice Community Consultation in the NSW Mining and Extractive Industries”, as developed by the NSW Minerals Council. Consultation with landholders is to include discussions on integrating any proposed mitigation works with the management of the property as a whole.

CONCLUSIONS

It is evident that the controls and restrictions on longwall extraction have been extended over the last ten years. This is largely evidenced by the conditions of development consents and the conditions attached to approvals issued under section 138 of the CMRA. With this growing awareness of subsidence has come a greater understanding of predicting the amount of subsidence and more importantly being able to identify the impacts, which subsidence will have on specific surface features. In the case of specific man made features the development of subsidence management plans has enabled coal to be recovered, which previously may have been sterilised. The obvious challenge is to develop the same style of approach and understanding for natural surface features. The requirement to develop SMP’s with an enhanced level of community and government agency liaison further supports the need for a total management approach.

Following on from possible limitations and restrictions on the way longwall mining is undertaken, is the need to predict with greater accuracy the level of subsidence resulting from a particular longwall layout. The amount of surface displacement, tensile and compressive strain, tilt and curvature must be predicted to an increasing level of accuracy. It is important to be able to categorically determine the degree of impact a particular amount of subsidence will have on a surface feature. Hence, not only must the measurement and prediction of subsidence be accurate but also the determination of the subsidence impacts must be correct. Failure to meet this challenge has the potential to cause the “right to mine” to be questioned and possibly revoked.

In conjunction with this need for more accurate subsidence prediction is the challenge to make each longwall panel and its related infrastructure more adaptive. Longwalls have traditionally been regarded as large inflexible “monsters” that require considerable planning and mining logistics. This is definitely the case for modern day longwall panels with face lengths of 300 metres or more. To manage the potential subsidence related impacts that arise from such longwalls, the mining operation must be able to provided localised protection to surface features by such means as varying face width, chain pillar sizes and installation and recovery locations. If the flexibility of longwall mining operations cannot be improved the potential resource sterilisation and resulting impacts of mine subsidence could drastically reduce the overall economic advantage the longwall mining method has over other extraction methods.
REFERENCES

Harvey C R, 1989 “Evaluation of Rock Formations in the Lithgow Region, with respect to coal mining”. Unpublished research project, University of New South Wales; pp 73

Holla L & Barclay E, 2000 “Mine Subsidence in the Southern Coalfields, NSW Australia” pp 118, NSW Department of Mineral Resources, Sydney.

NSW Govt, 1982 “Coal Mines regulation Act 1982” (NSW Government Printer; Sydney).


APPENDIX A

SUBSIDENCE MANAGEMENT PLAN REQUIREMENTS

In preparing an SMP and application for approval, the applicant shall:

- Identify properties and update ownership and land use with the area subject to the application;
- Fully describe the physical landforms and environment of the area, including water courses, wetlands, aquifers, water related ecosystems, forests, cliff lines and other sensitive areas, together with areas of potential conservation, heritage or archaeological significance;
- Survey drainage channels within and adjacent to the relevant area and fully describe base line surface and ground water flows and levels and water quality;
- Fully describe the inventory of surface infrastructure and other man made features within or adjacent to the area which is subject to the SMP, including but not limited to:
  - buildings (dwellings, offices, business premises, shed, etc);
  - sealed or gravel roads, access tracks other tracks, etc;
  - dams, bores, tanks, springs, water reticulation systems, etc;
  - on-site waste water systems, swimming pools, tennis courts etc;
  - service infrastructure and utilities (telecommunication lines, transmission lines, water sewage and other pipelines, etc);
- Assess current agricultural utilisation, agricultural improvements and the agricultural suitability of the area;
- Review current utilisation of the land for business purposes (other than agriculture, including the value of improvements and businesses;
- Provide comprehensive subsidence predictions, taking into account the results of any relevant previous subsidence monitoring undertaken and other factors such as topographic variations and geological complexities, with a description of the methodology and assessment of the reliability of the predictions;
- Provide detailed results of pre-mining base line monitoring of environmental values in areas of environmental sensitivity that may be damaged by subsidence (such as ground and surface water flows, water quality and water dependent ecosystems, based on at least a twelve month survey);
- Identify features that will potentially be subject to significant impacts resulting from subsidence and fully describe the expected impacts including:
  - surface water courses and ground water resources (impacts on water quality, river or ground water flows, and areas that will potentially be drained, inundated or affected by cliff falls, etc);
  - lake foreshores and flood prone areas;
  - other significant natural features, particularly cliffs;
  - significant ecological values;
  - major surface infrastructure;
  - other built structures and surface improvements;
  - known proposed surface developments;
  - surface features of community significance;
- Identify dwellings that may be subject to damage beyond safe, serviceable and repairable criteria;
- Identify agriculture or other businesses likely to be adversely affected;
- Identify and quantify the economic and social benefits of the proposed mine development (jobs, continued mine operation, regional economic development, royalties, etc) such as to provide sufficient information for balanced assessment;
- Investigate feasible mitigation and remediation measures that can be implemented to reduce and or rehabilitate subsidence impacts on significant natural features and ecological values;
- Identify the costs (production forgone, delays, added costs, etc) associated with various mitigation, remediation or surface feature protection options;
- Investigate other options if subsidence impacts cannot be reduced satisfactorily, such as compensation, acquisition, temporary relocation, or any other form of agreement with landowners and
- Identify all areas of potential compensable loss under the Mining Act 1992 and either reach agreement with landowners in regard to likely compensable loss, or determine suitable mitigation measures to minimise compensable loss.
CASE STUDIES IN THE APPLICATION OF INFLUENCE FUNCTIONS TO VISUALISING SURFACE SUBSIDENCE

Roger Byrne

ABSTRACT: The influence function method assumes that a subsidence surface can be represented as a mathematical function. In the case studies presented a gaussian function is used. Examples are given of how the influence function is calibrated to local mine performance, and also in greenfields applications. In the latter cases, geotechnical analysis of likely performance of pillars and roof and floor strata were incorporated into the analysis.

The major application of the influence function method is in visualising subsidence – as the method models surfaces and surface deformations. Detailed post mining topography and deformation histories can be produced.

INTRODUCTION

There is a range of methods used for predicting subsidence. These include:

• the prediction of two or three points on a cross line (for example Holla (1985) where $S_{max}$, inflection point and angle of draw are predicted)
• profile functions that predict the full shape of cross lines (for example the incremental profile method of Waddington and Kay (1998))
• influence functions that predict the shape of the subsidence surface, for example the Surface Deformation Prediction System (SDPS)

There is still no way to accurately predict subsidence solely from the application of geotechnical engineering to a knowledge of the rock mass, so all three methods require calibration to local experience when available. Holla (1988) and Waddington and Kay (2001) have discussed the validity of predictions and these authors suggest that predictions of vertical subsidence are within 10%. No study of the reliability of the predictions of tilt or strains have been conducted. The impact of unrecognised changes in geology are usually used to explain mis-predictions. Influence functions have not been extensively used to date so that there has been no study of the accuracy. However, since the method is calibrated in the same way as for the point and profile methods, a similar accuracy is likely.

Influence functions can be incorporated into integrated subsidence studies. Prediction is done separately using a combination of back-analysis of subsidence history and also the application of geotechnical engineering to determine panel sag, pillar compression, and roof/floor compression. This allows an assessment of the geology and hopefully improves the accuracy of the resulting predictions. Influence functions are then used to visualise the surface deformation that follows from the calibration.

FUNDAMENTALS

The shape of the subsidence profile across a panel or group of panels can be approximately described by mathematical functions. A number of different functions have been published. One of these, the bell-shaped Gaussian function is incorporated in the SDPS program, which Seeedsman Geotechnics Pty Ltd (SGPL) have used with success to visualise subsidence over various mine layouts.

Karmis et al (1990) describe the Gaussian function for the 2-dimensional case:

$$g(x,s) = \frac{(S_0(x))}{r} \cdot \exp\left(-\pi \cdot \frac{((x-s)^2)}{r^2}\right)$$
where:

\[ r = \frac{h}{\tan(\beta)} \]
\[ h = \text{the overburden depth} \]
\[ \beta = \text{the angle of principal influence} \]
\[ s = \text{coordinate of point P, where subsidence is considered} \]
\[ x = \text{coordinate of the infinitesimal excavated element; and} \]
\[ S_0(x) = \text{convergence of the roof of the infinitesimal excavated element} \]

Subsidence at any point can be expressed by the following equation:

\[ S(x, s) = \frac{1}{r} \int_{-x}^{+x} S_0(x) \exp\left(-\frac{\pi((x-s)^2)}{r^2}\right) dx \]

where:

\[ S_0(x) = m(x)a(x) \]
\[ m(x) = \text{extraction thickness; and} \]
\[ a(x) = \text{roof convergence (subsidence factor)} \]

The process of calibration requires fitting subsidence profiles calculated in SDPS to the profiles measured across the project. The subsidence basin calculated in SDPS, and hence the profiles across the panels in question are adjusted by changing the input parameters. Experience over a number of projects shows that the above functions usually produce acceptable results.

The calibration of the function to a specific project requires the following parameters:

- Subsidence factor (Smax /T ratio) as a percentage
- Location of inflection point (compensation width)
- Ratio of strain to curvature
- Angle of influence (angle to zero subsidence)
- Panel layouts
- Seam thickness
- Depth of cover

Influence functions are based on predicting surfaces and not profiles. Once a subsidence surface has been calculated, subsidence at specific points or along any line across the surface can be illustrated, by taking a slice through the gridded surface. To use the influence function, the locus of the inflexion points need to be supplied. For longwalls, this can mean that, at its simplest, four points need to be supplied to fully describe a panel.

Outputs possible from the influence function include:

- Vertical subsidence,
- Tilts (in east direction, north direction, and maximum),
- Curvatures (in east direction, north direction, and maximum),
- Strains (in east direction, north direction, and maximum),
- Horizontal movement (in east direction, north direction, and maximum)

Gridded surfaces of each of these parameters are obtained.

Run times for the program are very quick. Large projects with closed spaced prediction points will run in the space of minutes on a Pentium 3 or 4 computer with say 1.5GHz processing speed. For example a project with say 15 panels and prediction points at grid nodes of 10m (200,000 points) will run in the space of around 10-15 minutes. This quick run time allows fine tuning of calibrations against known data by allowing input parameters to be repeatedly changed until the closest possible calibration has been obtained.

Various case studies and the calibration results that have been gained for different projects are discussed.
SHALLOW LONGWALLS WITH NON-YIELDING CHAIN PILLARS

For shallow longwalls, where the chain pillar design is such that pillar deformation is low, each longwall panel can be considered independent from adjacent ones. The subsidence of the surface over a series of longwalls can be considered to be the result of the addition of the subsidence of each individual panel.

Figures 1 and 2 show the subsidence and strain measured across a line that traverses an extracted panel at a mine in the Hunter/Newcastle coalfield. The actual data is shown with the solid line and the calibration of the predicted surface is shown in the dashed line. The subsidence plot shows a good agreement between the actual and predicted. The profile has been well described by adjusting the location of the inflection point, the angle of influence and Smax/T. In this case subsidence over the pillar is represented well.

**FIG. 1 - Comparison of measured versus predicted subsidence at Mine A - Hunter/Newcastle coalfield**

Figure 2 shows the comparison of strains between measured and predicted. The actual measured data shows that maximum tensile and compressive strains are not well predicted in this case. This area shows localisation of strains, which is probably due to thin soil cover and widely spaced joints in the near surface rocks. There is a reasonable fit to tilts and curvatures for the data.

**FIG. 2 - Comparison of measured versus predicted strains at Mine A - Hunter/Newcastle coalfield**
VISUALISING THE SUBSIDENCE

Surface contours

Figure 3 shows a subsided topographic surface for Mine A in the Newcastle/Hunter Coalfield. Influence function methods were used to derive a grid of subsidence over the area. This subsidence grid was added to the grid of original topographic surface. The addition of the grid of existing R.L.’s of the original surface, plus the grid of negative subsidence values at each point results in the grid of predicted subsided topography. Once this grid has been calculated, it can be represented in a number of ways, including contour plan, coloured solid contour plan, and shaded relief plan. Slices of the gridded surface can be taken to show the profile in section. These sections can be straight lines between two points or can be more complex route lines. Examples include along surface infrastructure such as pipelines or along a creek that traverse the panels.

FIG. 3 - Predicted subsided topographic surface over Mine A – Hunter/Newcastle coalfield

Full movement of a transmission tower

Grids of subsidence, horizontal movement, strain and tilt, that had been calculated for Mine A were used to calculate the movement at the base of power transmission towers positioned above longwall panels. The change in movement at the surface at a point can be represented as curves showing movements as the panel retreats underneath it. Tower A is located approximately 37m from the centreline of a 229m wide panel. The panel is aligned at approximately 97°.
Figure 4 shows how tilt and subsidence change with the retreating panel. The curves show the subsidence and tilt when the longwall is approximately beneath the transmission tower. The panel start line is at approximately E300610m. Therefore the subsidence profile approaches zero at the Eastern end of the line, while the tilt profile mirrors the situation beneath the transmission tower.

**FIG. 4 - Tilt and subsidence at the location of a transmission tower, with changing longwall position**

Figure 5 shows the horizontal movement as the longwall retreats underneath the same tower. Up to 0.3m of horizontal movement is predicted.

**FIG. 5 - Horizontal movement above retreating longwall**
DEEP LONGWALLS WITH YIELDING CHAIN PILLARS.

At depths greater than about 250m, longwall chain pillars are typically designed to yield into the goaf. This allows increased reserve recovery and also reduces the amount of roadway driveage. Pillars may compress by up to 40% of their original height. The result is that eventually surface subsidence is dominated by the compression of the chain pillars and adjacent roof and floor strata and not the sag of the strata above the extraction panels.

As an example, Figure 6 gives an indication of the maximum vertical subsidence associated with 250m wide longwall panels and 30m pillars in a 3m seam as the depth is increased. Note that this would be an appropriate mine layout if there was a competent sandstone in the immediate roof, that allowed tailgate serviceability to be maintained. It is important to note that the chain pillars would be yielding under double goaf loading at a depth of around 250m. The figure shows that as depth increases, the proportion of subsidence contributed by pillar and roof and floor compression increases, while the proportion contributed by panel sag decreases.

![FIG. 6 - Contribution of various modes of subsidence with increasing depth](image)

In this model, the panel sag is taken from Holla curves (Holla, 1985), and roof and floor compression is based on rigid footing calculations (Poulos and Davis, 1974) with a Young’s modulus of 10 GPa. The pillar deformation is based on data from the Southern Coalfields (Figure 7) and an interpretation of laboratory data on model coal pillars (Figure 8) based on work by DAS (1986).

![FIG. 7 - Deformation data of Southern Coalfields coal pillars](image)
For the case of multiple panels and pillars where the depth of cover increases significantly such that the chain pillars yield when loaded, a different approach to subsidence prediction using SDPS is required. SDPS provides the ability to model each series of panels as one super panel. Contained in the boundaries of the super panels are chain pillars that act to reduce the subsidence. In this mode, SDPS models the negative subsidence (upsidence) of the pillars instead of the subsidence of the panels. This is depicted in Figure 9 where the super panel boundaries are shown in dotted line and the chain pillars in solid line.
The width of these active chain pillars is equal to the distance between the compensated panel boundaries. The feature of this method is that the pillars can be given their own negative subsidence factor, allowing greater control over subsidence prediction due to pillar compression. This can be observed in Figure 10 where the difference between the two series ‘-65% Pillar Subsidence Factor’ and ‘-55% Pillar Subsidence Factor’ – can be readily seen.

FIG. 10 - Comparison of 55% and 65% negative pillar subsidence factors - Mine B

Calibration

Mine B is in Central Queensland and has a thick weathering profile. Figure 10 shows the subsidence calibration and Figure 11 shows the strain calibration. A good calibration to Smax has been achieved. In addition, the shape of the strain curve shows a very good fit to the actual data. This may be due to the thick weathering profile and highly fractured bedrock, which reduces the likelihood of fracture localisation and hence strain peaks occurring.

FIG. 11 - Calibration of strain at Mine B
Visualisation of surface

The SDPS program allows large grids of subsidence predictions to be calculated quickly, in the order of minutes to seconds. This means that high data density grids are readily achievable. This density in turn allows the ability to display in high resolution graphics. Figure 12 shows the predicted subsidence over Mine C in Central Queensland, displayed as a line contour plan.

FIG. 12 - Subsided topography prediction for Mine C in Central Queensland

Queries can be run along a line through the subsided topographic surface. This allows the subsidence strain to be illustrated along infrastructure routes, for example a cable line, or along creek lines. Figure 13 shows the existing levels and subsided levels along a creek for Mine D.

FIG. 13 - Comparison of existing topography and predicted subsided Topography along the route of a creek
Figure 14 shows a shaded profile surface of the subsided topography over Mine C. The subsidence troughs and humps are clearly visible in the surface (angled panels in the SE part of the figure). This is a useful way of presenting data to non-technical persons, with the important note that the surface has been exaggerated to show the subsidence basins. Different amounts of exaggeration can be applied.

**FIG. 14-** Shaded relief surface of subsided topography

**CONCLUSIONS**

The use of influence functions is an efficient and flexible method for generating subsidence parameters over surfaces. Calibration of the function against measured subsidence is essential, and allows a measure of reliability of the predictions. Subsidence prediction is undertaken by using geomechanical and empirical methods to determine the behaviour of pillars and strata. These methods generate the input parameters to the influence function program. The program generates surfaces very quickly allowing repeated calculation runs to fine tune calibrations against measured data.

Once confidence in the input parameters has been gained from the calibration, then the program can be used to visualise the surface deformations. The program generates a surface rather than just a point or line profile across the surface. This allows a detailed view of the subsidence basin. Sections across the area, routes along the surface and specific points can be queried from the generated deformation grids. Surface gridding programs can be used to produce a variety of graphical outputs. This flexibility in graphical output makes the program very useful for a range of end users, from technical analysis through to non-technical viewing of the predicted subsided surface.

The SDPS influence function program was developed in the US, and has been successfully used in a number of projects in the Eastern US coalfields since the late 1980’s. Over the past few years, the combination of geotechnical analysis to predict subsidence, plus the use of SDPS for visualisation of the deformed surface has been used successfully on a number of Australian coal projects. The method has great potential to help meet the increasing demands that are currently being placed on subsidence prediction.
REFERENCES


Karmis M, Agioutantis, Z and Jarosz, A 1990 Recent developments in the application of the influence function method for ground movement predictions in the US. Mining Science and Technology, 10, 233-245.


THE IMPACTS OF MINE SUBSIDENCE ON CREEKS, RIVER VALLEYS AND GORGES DUE TO UNDERGROUND COAL MINING OPERATIONS

Arthur Waddington ¹ and Don Kay ¹

ABSTRACT: Measured subsidence profiles above coal mining operations regularly show less than expected subsidence at creeks, river valleys and gorges. Horizontal measurements across such surface notches indicate that they reduce in width as mining occurs. The reduction in subsidence is referred to as 'upsidence' and the reduction in width is referred to as valley closure. The upsidence and closure movements tend to increase in amplitude as the size of the valley increases, and as the magnitude of subsidence increases. The movements are greatest when the in situ horizontal stresses are high and when the valleys are fully undermined. The upsidence is a combination of anticlinal valley bulging and buckling or shearing of the surface and near-surface strata.

As Longwalls 8 and 10, at Tower Colliery, in the Southern Coalfield of New South Wales, were mined beneath the Cataract River Gorge, the incremental upsidence in the base of the gorge, due to mining each longwall, was approximately 360 mm, resulting in the base of the gorge being uplifted as much as 250 mm above its original level. At the same time, the width of the gorge was reduced by approximately 280 mm. Cracking and buckling of the strata, within the base of the gorge, resulted in a loss of water from some of the natural ponds in the bed of the river, with consequential criticism from local landholders and regulatory authorities.

A comprehensive research study supported by an ACARP grant and assisted by CSIRO and The University of New South Wales has provided some additional insight into the valley bulging phenomenon. The major findings arising from the research project provide new methods for the prediction of mining-induced ground movements in creeks, river valleys and gorges.

INTRODUCTION

The major findings of a research project, funded by ACARP research grants, which was carried out between March 1999 and September 2002 have been summarised. The research work was funded in two halves, as ACARP Research Projects C8005 and C9067. Detailed research reports on these projects, (Waddington and Kay 2001d, 2002), are available on CD and copies can be obtained from the offices of Australian Research Administration Pty Ltd (ARA) in Brisbane (Tel: 07 3229 7661). The research reports form the basis of the ‘Management Information Handbook on the Undermining of Cliffs, Gorges and River Systems’, for the mining industry, which is also available from ARA.

ANALYSIS OF MEASURED GROUND DISPLACEMENTS

When longwalls are extracted beneath steeply incised terrain, the ground movements that occur around the longwalls are very complex, particularly within a high horizontal in situ stress regime, and these complex movements result from a number of distinct mechanisms. The measured movements can be a combination of some or all of the following components:

- Normal mining-induced horizontal movements of points on the surface around an extracted panel, as subsidence occurs, which are generally directed towards the centre of the extracted goaf area.
- Upsidence and closure of creeks, gullies, river valleys and gorges due to valley bulging, which is caused by redistribution of pre-existing in situ stresses in the base and walls of the valley, as mine subsidence occurs.
- Predominantly horizontal displacements of surface strata due to release and redistribution of pre-existing regional, or far-field, in situ stresses, as the extracted goaf areas increase in size within a local mining area.

¹ Waddington Kay & Associates
• En masse slippage movements in a downhill direction due to topographic factors.
• Differential movements of the strata on opposite sides of a fault line.
• Continental drift, which is known to change the positions of points on the Australian Plate by approximately 70 mm each year towards the northeast.

In order to develop methods for the prediction of each of the above components of movement, the measured data, ideally, have to be broken down into the various components prior to analysis. This is not, however, an easy task, because in most cases the measured survey movements are relative movements rather than absolute movements. When analysing the closures that have been measured in creeks and river valleys due to valley bulging, however, it appears that many of the other components have little or no effect on the closure measurements.

En masse slippage down steep slopes, due to mining is a relatively rare occurrence and is due to the instability of surface soils in particular locations. Where steep slopes exist and could be affected by mining it would be prudent to study the geology of the site and the nature of the surface soils so that any unstable areas can be identified.

It is possible that some of the data which formed the basis of the research projects discussed in this paper could have been affected by this mechanism, but if so it will have led to overstatement of closure movements.

In analysing the valley closure data, no allowance was made for differential movements caused by regional horizontal stress redistribution or continental drift, because the differential movements in the two sides of a valley, as a result of these mechanisms, would be relatively small and in many cases negligible.

In the steep-sided Cataract and Nepean River Gorges it was found that the closures in the sides of the gorges were almost mass movements with little differential shear displacement between different horizons in the strata. Almost all of the closure, therefore, occurred in or just below the base of the gorge. Because the gorge bases were relatively narrow, the differential mining-induced horizontal movements, due to differential tilting in the sides of the gorges, were relatively small in comparison with the closure movements.

In the vee-shaped valleys, a large proportion of the closure occurred in the bottoms of the valleys, coupled with localised concentrations of compressive strain, but some of the closure was noted to occur at horizons above the bottom of the valley. This observation from measured data is supported by numerical modelling results, which indicate that in vee-shaped valleys some of the shearing occurs along weaker horizons in the valley sides. The closure movements are, therefore, spread over a greater width than those measured in the gorges. It is, therefore, possible that some of the measured closure data in the vee-shaped valleys could have been affected by differential mining-induced horizontal movements in the valley sides.

In some cases these differential movements could have caused the sides of the valley to open and the measured closure, being the sum of the two movements, could, therefore, be less than the actual closure caused by valley bulging.

The extent to which the data might have been affected in this way is difficult to determine, because many of the surveys that were carried out in the past did not measure the absolute movements of the ground in three dimensions and the closures, in those cases, have been calculated from the strains.

The method that has been developed for the prediction of closure is, therefore, based upon the overall closure of the valley recognising that, in the case of vee-shaped valleys, some of the movement will occur in the valley sides.

When predicting closures in vee-shaped valleys it would be prudent to ignore the impacts of differential mining-induced horizontal movements in the valley sides, if those movements cause a reduction in the predicted closures.

**NORMAL MINING INDUCED HORIZONTAL GROUND MOVEMENTS**

In flat or gently sloping terrain, i.e. where steep slopes or surface incisions do not influence ground movement patterns, the subsidence induced horizontal displacements are generally directed towards the centre of the mined...
longwall panel. The ‘normal’ horizontal component of subsidence, also referred to as horizontal displacement, can be determined at a point, approximately, by multiplying the tilt at that point by an appropriate strain-curvature factor. Predicted subsidence profiles can be obtained using the Incremental Profile Method, (Waddington and Kay 1995, 1998a, 198b, 2001b), or other methods calibrated to local data. Predicted tilt profiles can be determined as the first differentials of the predicted subsidence profiles.

The appropriate strain-curvature factor for the Southern Coalfield is 15 and if, for example, the predicted tilt at a point is 2 mm/m, then, the predicted horizontal ground displacement will be approximately 30 mm towards the centre of the mined goaf. The appropriate strain-curvature factor for the Newcastle Coalfield is 10 and if, for example, the predicted tilt at a point is 2 mm/m, then, the predicted horizontal ground displacement will be approximately 20 mm towards the centre of the mined goaf.

Whilst this method is only approximate, it tends to be conservative where the tilts are high and tends to understate the horizontal movements where the tilts are low. Where the tilt is low, the ‘normal’ horizontal displacement is generally very small, even though it could be many times greater than the vertical subsidence at the same point.

The tilts reduce with increasing distance from the goaf edge of the longwall. At the edge of the subsidence trough, where the tilts approach zero, any small horizontal displacement at that point could be infinitely greater than the tilt.

When large horizontal displacements are measured outside the goaf area, they are more likely to be caused by regional movements, as discussed in later.

VALLEY BULGING EFFECTS DUE TO MINING BENEATH GORGES, RIVER VALLEYS AND CREEKS

When creeks and river valleys are affected by mine subsidence, the observed subsidence in the base of the creek or river is, generally, less than the level that would normally be expected in flat terrain. This reduced subsidence is due to the floor of the valley bulging and buckling upwards. This phenomenon is referred to as valley bulging and is caused by the redistribution of, and increase in, the horizontal stresses in the strata immediately below the base of the valley as mining occurs. Valley bulging is a natural phenomenon, resulting from the formation and ongoing development of the valley, but the process is accelerated by mine subsidence. The phenomenon appears to be triggered, to varying degrees, whenever escarpments, gorges, river valleys, creeks or other surface incisions are undermined.

The graph in Figure 1 shows a series of typical subsidence profiles along a survey line that crosses a tributary of Brennans Creek and an upsidence spike is clearly visible, coincident with the base of the valley. The upsidence was accompanied by closure as indicated by the bay length differences shown in the graph.

![FIG. 1 - Typical Subsidence Profiles across a Valley showing an Upsidence Spike](image-url)
The local reduction in subsidence, which is referred to as ‘upsidence’, is generally accompanied by localised changes in tilt and curvature leading to high compressive strain in the centre of the valley and horizontal closure of the valley sides. In the case of escarpments and wide river gorges the movements may be limited to the cliffs that are closest to the extracted area.

In most cases studied, the upsidence effects extend outside the valley and include the immediate cliff lines and the ground beyond them. For example, monitoring within the Cataract Gorge, at Tower Colliery, as Longwalls 8 and 10 were mined, revealed that the upsidence extended up to 300 metres from the centre of the Gorge, on both sides of the Gorge. In that case, the magnitude of the upsidence movements was greater than the subsidence leading to an overall uplift in the base of the Gorge, consequently leaving it above its original pre-mining level.

In other cases, within creek alignments, upsidence has been observed well outside an extracted panel, apparently due to a beam within the near-surface strata rotating and pivoting as a seesaw, as one end of it rises and the other subsides. However, in these cases, the measured upsidence and strains have been less than would be expected to arise from the compressive buckling mechanism described above.

During the first stage of the research project, many cases of valley bulging were analysed and it was concluded that the magnitude and lateral extent of the upsidence and the extent of the closure appeared to be dependent on many factors including:

- the depth of the valley,
- the width of the valley,
- the shape of the valley,
- the direction and magnitude of in-situ horizontal stresses,
- the rock strengths and fracture characteristics,
- the local stratigraphy and joint spacing and
- the magnitude of the mining induced stresses, which are dependent on;
  - the cover depth,
  - the seam thickness,
  - the panel and pillar widths and
  - the location of the valley, or escarpment, relative to the goaf edges.

Based upon the empirical evidence, upsidence and closure movements can be expected in cliffs and in the sides of valleys, whenever longwalls are mined beneath them. Such movements, however, tend to be smaller outside the goaf areas and tend to reduce as the distance outside the goaf edge increases. The movements are incremental and increase as each longwall is mined in sequence and the movements caused by the mining of one longwall can be spread over several longwalls.

During the second stage of the research project additional cases of closure and upsidence were studied and this led to the conclusion that the major parameters affecting the closure and upsidence were:

- The lateral distance from the base of the valley to the side of the current longwall.
- The longitudinal distance from the base of the valley to the end of the current longwall.
- The depth of the valley.
- The maximum incremental subsidence over the current longwall.
- The direction and magnitude of the in-situ horizontal stress.

Based upon this finding, methods of prediction have been developed for closure and upsidence, as discussed below.

THE PREDICTION OF CLOSURE IN CREEKS AND RIVER VALLEYS

A method for the prediction of closure in creeks and river valleys is based upon measured data over a wide range of cases, with valley depths varying from 27 metres to 74 metres. The data was mainly collected from collieries in the Southern Coalfield where the valleys are incised into flat lying sedimentary deposits and where the in situ horizontal stresses are high. The method of prediction would be expected to give best results in areas with similar geology and similar stress regimes. The method is based upon upper-bound measured values and it is anticipated...
that it will overpredict in areas of lower stress. Further research is required to determine how pre-existing in situ horizontal stress influences the closure movements.

The method for the prediction of closure is based upon a series of graphs that show the interrelationships between closure and a number of contributory factors. The interrelationships between the factors are illustrated in Figures 2 to 5.

- Figure 2 shows the graph of closure plotted against the transverse distance from a point in the bottom of the valley to the advancing goaf edge of the longwall divided by the width of the panel plus the width of the pillar.

![Graph of Valley Closure versus Transverse Distance from the Advancing Goaf Edge](image)

- Figure 3 shows a longitudinal distance adjustment factor plotted against the longitudinal distance from a point in the bottom of the valley to the nearest end of the longwall in metres.
- Figure 4 shows a valley depth adjustment factor plotted against valley depth.
- Figure 5 shows an incremental subsidence adjustment factor plotted against the maximum incremental subsidence of the panel.

The graphs indicate the upper bound values, which are mainly based upon closure data from the Cataract and Nepean Gorges, where the maximum incremental subsidence was approximately 410 mm and the depth of gorge was approximately 68 metres.

The closure is initially predicted from the upper-bound line of the graph shown in Figure 2 and the value so obtained is multiplied by the factors obtained from the graphs shown in Figures 3, 4 and 5, depending on the position of the bottom of the valley relative to the end of the longwall, the valley depth and the maximum incremental subsidence of the longwall.

The valley depth is determined by calculating the average level of the opposite sides of a valley and then deducting the level in the bottom of the valley.

The depth is easy to define in the case of a valley or gorge that is incised into an otherwise flat plain, but it is not so easy to define where the surface is undulating. In this situation, the valley depth has to be calculated relative to the average surface level.
In wide vee-shaped valleys, the sides of the valley have been defined as points on each side of the valley that are located at a horizontal distance of half the depth of cover from the lowest point in the bottom of the valley.

In all cases, the closure is measured at right angles to the general alignment of the valley.

FIG.3- Valley Closure Adjustment Factor versus longitudinal Distance
Figure 6 shows the distance measurement convention used to define the location of the point for which closure and subsidence predictions are required.
FIG. 6 - Distance measurement convention for closure and upsidence predictions

The transverse distances plotted in Figure 2 are the distances measured at right angles to the advancing goaf edge of the longwall expressed as a proportion of the width of the panel plus the width of the pillar.

For example, the transverse distances for points A, B, C and D in Figure 6 are -270 metres, 115 metres, 460 metres and 680 metres, respectively, distances outside the goaf being negative.

The longitudinal distances plotted in Figure 3 are the distances from the nearest end of the longwall, measured parallel to the longitudinal centreline of the longwall.

For example, the distances for points A, B, C and D in Figure 6 are 450 metres, 350 metres, 160 metres and -130 metres, respectively, distances outside the goaf again being negative.

THE PREDICTION OF UPSIDENCE IN CREEKS AND RIVER VALLEYS

A method based upon measured data over a wide range of cases, with valley depths varying from 8 metres to 87 metres can be used for the prediction of upsidence. The data was collected mainly from collieries in the Southern Coalfield where the valleys are incised into flat lying sedimentary deposits and where the in situ horizontal stresses are high. The method of prediction would therefore give better results in areas with similar geology and similar stress regimes. The method is based upon upper-bound measured values and it is anticipated that the method will overpredict in areas of lower stress. Further research is required to determine how pre-existing in situ horizontal stress influences the upsidence movements.

The method for the prediction of upsidence is the same as the method for the prediction of closure and is based upon a series of graphs that show the interrelationships between upsidence and a number of contributory factors. The interrelationships between the factors are illustrated in Figures. 7 to 10.

- Figure 7 shows the graph of upsidence plotted against the transverse distance from a point in the bottom of the valley to the advancing goaf edge of the longwall divided by the width of the panel plus the width of the pillar.
- Figure 8 shows a longitudinal distance adjustment factor plotted against the longitudinal distance from a point in the bottom of the valley to the nearest end of the longwall in metres.
- Figure 9 shows a valley depth adjustment factor plotted against valley depth.
- Figure 10 shows an incremental subsidence adjustment factor plotted against the maximum incremental subsidence of the panel.
FIG. 7 - Graph of Upsidence versus Distance from the Advancing Goaf Edge of the Longwall relative to the Width of the Panel plus the Width of the Pillar

Observed data points
Adjusted data
(all adjs except lateral distance)
Adjusted Upper Bound Curve
(all adjs except lateral distances)

FIG. 8 - Graph of Upsidence Adjustment Factor versus Longitudinal Distance

Observed data points
Adjusted data points
(all adjs except no longit. ends)
Adjusted Upper Bound Curve
(all adjs except no longit. ends)
FIG. 9 - Graph of Upsidence Adjustment Factor versus Valley Depth

FIG. 10 - Graph of Upsidence Adjustment Factor versus Maximum Incremental Subsidence
The graphs indicate the upper bound values, which are mainly based upon upsidence data from the Cataract Gorge, where the maximum incremental subsidence was approximately 350 mm and the depth of gorge was approximately 70 metres.

The upsidence is initially predicted from the upper-bound line of the graph shown in Figure 7 and the value so obtained is multiplied by the factors obtained from the graphs shown in Figures 8, 9 and 10, depending on the position of the bottom of the valley relative to the end of the longwall, the valley depth and the maximum incremental subsidence of the longwall.

The transverse distances plotted in Figure 7 are the distances measured at right angles to the advancing goaf edge of the longwall expressed as a proportion of the width of the panel plus the width of the pillar. Figure 6 shows the distance measurement convention used to define the location of the point in the creek for which closure and upsidence predictions are required.

The longitudinal distances plotted in Figure 8 are the distances from the nearest end of the longwall, measured parallel to the longitudinal centreline of the longwall.

**THE LATERAL DISTRIBUTION OF UPSIDENCE**

Upsidence in a valley is the result of two separate mechanisms, namely an anticlinal valley bulging coupled with buckling or shearing of the strata in the base of the valley. The maximum upsidence occurs in the bottom of the valley, where the buckling or shearing effect occurs, but the valley bulging effect spreads outwards from the bottom of the valley under both sides of the valley for a considerable distance. For example, in the Cataract Gorge above Longwall 8 at Tower Colliery, whilst the upsidence in the base of the gorge was 360 mm, the upsidence in the clifflines was around 100 mm and the upsidence effect extended for a distance of 300 metres on each side of the gorge.

The upsidence profile is dependent upon the way in which the rocks in the bottom of the valley buckle upwards and, since this is controlled by local geology, any method for predicting the profile can only be expected to provide approximate answers.

Figure 11 shows idealised profiles of upsidence across the Cataract Gorge, both along the goaf edge and along the centreline of the longwall.

**FIG. 11 - Idealised Upsidence Profiles across the Cataract Gorge**
It can be seen that the lateral spread of the upsidence was greater where the amplitude of the upsidence was greater. Further research is required in order to develop a more definitive method for the prediction of upsidence profiles, but in the meantime it seems reasonable to model the profiles on the upper measured profile shown in Figure 11. An approximate profile can be obtained by scaling both the width and amplitude of the profile in proportion to the predicted upsidence value. It should be noted, however, that the predicted profile can only be approximated since the actual buckling will depend upon local geology and might not be centrally positioned in the bottom of the valley or gorge.

THE ANALYSIS OF COMPRESSIVE STRAINS IN CREEKS AND RIVER VALLEYS

The relationship between the closure in the sides of a valley and the maximum measured horizontal compressive strain in the base of a valley is complex, because it is very much dependent upon local geology. It seems reasonable to assume, however, that as the closure increases, the level of strain should increase and this appears to be borne out when measured data is studied.

Figure 12 shows a graph of closure, across the steep section of a valley, versus maximum compressive strain, based upon data that was observed over longwalls at Tower, Appin, West Cliff and Baal Bone Collieries. The trend for strains to increase as closures increase is clearly seen.

The data includes measured strains from surveys with different bay lengths, which is the cause of some of the scatter in the data points shown in the graph. The curved line drawn on the graph indicates the upper-bound limit of the data for bay lengths of 20 metres. The straight lines show what the relationship between strain and closure would be for bay lengths of 10 metres and 20 metres, if the closure all occurred in one bay.

FIG. 12 - Graph of Strain versus Closure in the Steepest Part of the Valley

The upper bound graph can be used to predict the likely maximum level of compressive strain in the base of the valley, over a bay length of 20 metres, once the predicted overall closure of the valley has been determined. The strain values obtained from the upper bound graph are generally expected to be conservative and are unlikely to be exceeded in practice.

The likely range of strains can also be determined from the graph for particular values of closure and it can be seen that the range can vary considerably. This is due to variations in the nature of the topography and the levels of insitu horizontal stress in the base of the valley.
It might at first seem paradoxical that, in some cases, the predicted overall valley closure is less than the closure in the base of the valley, over a 20 metre bay length, based on the predicted strain. This is because the predicted closure is the overall closure of the steepest part of the valley and as the closure occurs in the base of the valley, due to compressive failure of the bedrock, the sides of the valley expand due to stress relief.

This situation is illustrated in Figure 13, where the overall valley closure is shown as 100 mm, whilst the closure in the base of the valley is 160 mm. The closure in the base of the valley should thus be calculated from the predicted strain over a survey bay length of 20 metres.

Similarly, the overall closure in the base of a gorge can be greater than that measured over a 20 metre survey bay length and greater than that measured overall between survey marks behind the cliff lines on the plateau. This is illustrated in Figure 14, where the overall closure is 100 mm, whilst the closure in the base of the gorge is 350 mm. The maximum strain of 12 mm/m occurs over a 20 metre survey bay length in the base of the gorge.

FIG. 13 - Typical Closure Movements in a Vee-Shaped Valley

FIG. 14 - Typical Closure Movements in a Steep-Sided Gorge
MINING INDUCED REGIONAL OR FAR-FIELD HORIZONTAL GROUND MOVEMENTS

In addition to the ‘normal’ and valley related movements, far-field regional movements have also been recorded in a number of cases, at considerable distances from the mined goaf areas. Such movements have often been several times higher than the vertical subsidence movements measured at the same points.

It has been conjectured that these regional movements are caused by redistribution of the stresses in the strata between the seam and the surface due to the regional mining activity. The direction of such movements would tend to be towards the currently active mining area, but the direction of movement could also be dependent upon the scale and proximity of previous mining in adjacent areas.

It has been suggested by some authors that the regional movements are generally aligned with the principal horizontal in situ stress direction and initially this appeared to be the case in the Cataract Gorge over Longwalls 8 to 17 at Tower Colliery. It seems, however, more reasonable to suggest that the movements will be directed towards areas where the confining stresses have been reduced by mining activity, thus allowing expansion of the strata to occur.

The stresses generally within the strata are compressive in all directions and until mining occurs the stresses are in equilibrium, the balance being controlled by the shear resistance within and between strata units. As mining occurs, the equilibrium is disturbed and the stresses have to achieve a new balance by shearing through the weaker strata units and by expanding into areas of greatest dilation, i.e. towards the goaf areas, where the confining stresses have been relieved.

THE PREDICTION OF REGIONAL HORIZONTAL MOVEMENT

A method for the prediction of regional horizontal movements has been developed. The method is based upon measured data over a wide range of cases, with measured horizontal displacements up to 125 mm and distances from the mined goaf up to 6.2 times the depth of cover. The measured displacements were found to be greater where the in situ horizontal stress at seam level was greater. The measured displacements increased incrementally as each longwall was mined and the greatest displacements occurred as the total width of extraction became critical.

![Graph showing Incremental Regional Horizontal Movements plotted against Horizontal Distances from Goaf Edge divided by Depth of Cover](image-url)

FIG. 15 - Graph showing Incremental Regional Horizontal Movements plotted against Horizontal Distances from Goaf Edge divided by Depth of Cover
It can be seen from the data in Figure 15 that the regional movement is greatest at the goaf edge and decreases with increasing distance from the goaf edge. The maximum recorded movement was almost 125 mm and movements were recorded as far away as 6.2 times the depth of cover from goaf edge, i.e. at a distance in excess of 3 kilometres. The movements are maximum incremental movements caused by the mining of one longwall and the upper-bound data relates to the second longwall in each series, which caused the highest movements.

The top curve in the graph is the upper-bound curve for the data from Tower Colliery, where principal in situ horizontal stresses as high as 44 MPa have been measured, at seam level. The bottom curve is the upper-bound curve for the data from South Bulli Colliery where the maximum principal horizontal in situ stress measured at seam level was approximately 26 MPa. It therefore appears that the regional horizontal movements are almost proportional to the in situ horizontal stress at seam level.

The curves can be used with care to predict the regional movements that are likely to occur, due to mining longwalls at collieries in the Southern Coalfield, once the in situ stresses at seam level have been determined. The maximum movements tend to occur when the second and third longwalls are mined in a series and to decline as subsequent longwalls are mined. This is possibly due to the fact that once the strata has been stress relieved by the first few longwalls, the potential for further movement is reduced.

Research in this area is continuing and it is hoped that a greater understanding will be gained by further study.

CONCLUSIONS

The research carried out by the authors during the last three years has provided considerable insight into mining-induced movements in the vicinity of creeks, river valleys and gorges. Useful methods have been developed to enable the prediction of valley closure, valley upsidence, compressive strain and regional horizontal movement.

It should be noted that the predictive methods are based upon data measured in the Coalfields of New South Wales, where the in situ horizontal stresses are generally high and, since they use upper-bound values, are expected to overpredict at collieries where the in situ stresses are lower than at Tower Colliery. Further research is required to investigate the actual influence of in situ horizontal stress on the mining-induced movements. It seems reasonable to assume that the movements, which are caused by the in situ horizontal stresses in the strata, would be proportional to the pre-mining levels of in situ stress. At this stage, however, this can only be conjectured, since baseline data is not available to support this assumption.

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- NSW Dams Safety Committee
- Sydney Catchment Authority
- Baal Bone Colliery
- Angus Place Colliery
- South Bulli/ Bellambi Colliery

REFERENCES


ABSTRACT: Following a series of highly publicised environmental issues relating to subsidence of rivers, streams, lakes and other surface water bodies, as well as groundwater systems, the longwall coal mining approval and environmental management process in NSW now has a significant focus on predicting, monitoring, managing and rehabilitating adverse effects on surface water and groundwater systems.

Due to the recent public and regulatory focus on the issue, future longwall layouts and approvals may be required to demonstrate that unacceptable effects will not be imposed on rivers and streams. This may result in significant changes to panel layouts, or if no acceptable management and rehabilitation measures are available, the worst case scenario may occur where an environmentally sensitive water body may not be undermined.

The range of environmental effects that have been directly observed by the author, as well as reported effects, or effects that could be anticipated from subsidence in NSW coalfields are outlined. Measures required to assess the potential effects on environmental systems, potential rehabilitation requirements, potential risk assessment and management procedures relevant to water related issues are discussed.

The effects of longwall subsidence are outlined for a range of geomorphological areas, whilst an indicative process that can be used to assess the potential severity of effects on surface water and groundwater systems, which leads to the degree of attention required to manage them, are described.

INTRODUCTION

Currently, environmental issues relating to subsidence are managed by approvals under either Section 138 of the Coal Mines Regulation Act 1982, and/or development consent attained through the Environmental Impact Statement process.

Significant changes in community and regulatory attitude requires longwall mines to improve the environmental management of subsidence to conform to NSW Government interagency requirements, as well as concerns of the wider community and other stakeholders. This has generated an enhanced need for responsible resource recovery, with a focus on the reducing impacts on surface water and groundwater systems, improving land use “functionality” and reducing adverse effects on the environment and local communities.

The New South Wales Department of Mineral Resources (DMA) is in the process of formalising and initiating a new longwall extraction approval and management process through “Subsidence Management Plans” (SMP), which will, in turn, replace the current S138 system. The SMP process will involve a greater focus on community consultation, interagency (Department of Land and Water Conservation (DLWC), Environmental Protection Agency (EPA), National Parks and Wildlife Service (NPWS), Planning NSW, SCA, NSW Fisheries) and NGO involvement in approving extraction of coal by longwall operations.

The SMP system will be implemented for the life of the approval (potentially up to seven years), and will involve issues such as ongoing community consultation, regular reporting to the DMR and potential revision of operating approvals and conditions as the mine proceeds. The SMP will involve a greater focus on improved subsidence prediction, environmental risk assessment and risk prioritisation in regard to surface water and groundwater that may be adversely affected by subsidence.

1 Geoterra
The following surface water and groundwater issues may potentially be addressed:

- Incorporating community concerns during mine planning and approvals process, as well as ongoing community consultation
- Development of risk management plans for sensitive or important environmental features
- Enhanced monitoring, reporting and review programmes
- Incorporation of monitoring, mitigation and rehabilitation of subsidence effects in mine planning, as well as operational and post mine closure situations

The SMP process will require characterisation of natural features such as streams, rivers, groundwater systems, wetlands, swamps, lakes or escarpments along with at least one years baseline monitoring for relevant issues in environmentally sensitive areas.

In addition, non “environmentally” sensitive areas that may be affected, such as farm land will also require assessment of potential changes to the “functionality” of the land. Functionality refers to land holder concerns regarding operating on the land, such as access to creek crossings or groundwater supplies, as well as ponding and erosion of stream bed and banks, cattle access to water, damage to fences and gates, changes in drainage patterns to name a few.

Water related issues to be addressed in a SMP are:

- land use property identification and ownership;
- inventory of watercourses, wetlands, aquifers, water-related ecosystems, forests, cliff lines and other sensitive features in the subsidence area;
- survey of drainage channels within the mine subsidence area and description of base line surface water and groundwater systems and water quality;
- inventories of surface infrastructure and other man-made features within or adjacent to the area which is subject to the SMP, including but not limited to dams, bores, tanks, springs, water reticulation systems, on site waste water systems, swimming pools, and sewerage systems;
- comprehensive subsidence predictions of the relevant surface water body, taking into account topographic variations and geological complexities;
- pre-mining base-line monitoring of ground and surface water flows, water quality and water dependent ecosystems, based on at least a twelve-month survey;
- assessment of potential impacts on water quality, river or groundwater flows and areas that will potentially be drained, inundated or affected by cliff falls;
- descriptions of lake foreshores and flood prone areas;
- outlining significant ecological values of surface water and groundwater systems in the area;
- agricultural or other businesses that may be adversely affected;
- feasible mitigation and remediation measures to reduce and/ or rehabilitate subsidence impacts on significant natural features and ecological values;
- costs (including production foregone, delays, and added costs) associated with various mitigation, remediation or surface feature protection options;
- investigation of other options if subsidence impacts cannot be reduced satisfactorily, such as compensation, acquisition, temporary relocation, or agreement with landowners; and
- identification of all areas of potential compensable loss under the Mining Act 1992 and either reach agreement with landowners in regard to likely compensable loss, or determine suitable mitigation measures to minimise compensable loss.

SURFACE WATER

There are a number of water related features and issues that may be affected by longwall subsidence.

Loss of Surface Water

The dominant environmental impact noted from subsidence is the reduction in surface flows or changes in surface water depths over subsided rivers, streams, upland swamps, wetlands, lakes or farm dams. The loss occurs due to cracking the bedrock beneath a surface water body, with the severity depending on a wide range of associated factors, such as the;
ephemeral or perennial nature of the water body, with a lesser chance of observable loss during dry periods;

transfer of surface water into the underlying mine, particularly for mines less than 100m to 130m below the surface. Some mines in the Newcastle field use the algorithm of 45T + 10m (T = seam thickness) to assess if there is a potential for interconnection of a surface water body and the underlying mine. A 2.5m thick seam, for example, could potentially have surface water inflows for a depth of cover less than 122.5m;

location of the subsidence area in the catchment, with lower order streams (which are higher in the catchment) generally having a less observable water loss compared to higher order, lower catchment streams;

position of the stream or water body over the tensile or compressive zones of the subsidence trough. Greater water losses occur over the tensile zones at the outer edges of subsidence troughs due to the greater development and interconnection of cracks.

depth and type of sediment in a stream, wetland, swamp, lake or dam bed. Greater depths of clayey content soils have a higher capacity to self seal the cracks, compared to shallow sandy, or exposed rock at the base of a water body

mineralogical nature of the sediment. Sediments with a higher clay content and lower dispersivity have a better potential to seal cracks underneath a surface water body compared to higher permeability, sandy sediments;

outcrop of bedrock. Areas of outcropping bedrock have a higher potential for surface water loss as the orders of magnitude increase in bedrock permeability due to subsidence cracking, is not reduced by infilling from sediments

relative inflow to the subsided zone compared to the hydraulic conductivity of the sediment / bedrock. Total loss of surface water will occur where a surface water body has an inflow rate below the volume that can flow out through the cracks. A range of variations from total to no loss can occur, depending on the relative inflow and outflow losses to the system.

Losing or gaining streams. A "losing" stream is where the surface water is higher than the regional groundwater table, and the water “falls” into the regional water table from the stream or other perched water body. A gaining stream is one that is recharged by groundwater inflow from the surrounding catchment. Losing streams are more susceptible to surface water losses through subsidence cracking of their substrate, whereas gaining streams may have an enhanced (although of limited duration) flow due to the higher imposed permeability of the catchment draining into the stream or other body.

Reduced Surface Water Supplies

Consumers relying on a surface water body for their water supply can be deleteriously affected if it, at worst, dries up, or to varying degrees, is reduced.

Reduced Water Quality

The reduction in water quality is generally not a significant issue for most mines, except within the Sydney Catchment Authority areas in the Southern and Western Coalfields.

Generally an increase in iron hydroxides can change the water appearance by varying degrees to an orange brown colour. This effect is due to enhanced shallow groundwater flow through fresh fractures partially dissolving iron sulfides (generally marcasite) and iron hydroxides within the fresh bedrock and precipitating the iron at chemical redox phase changes, such as where chemically reduced groundwater with dissolved iron species flows into an oxidising surface water body. The effect is generally more important from an aesthetic rather than chemical viewpoint, however the dissolution of iron minerals can also liberate low concentrations of associated metals. This dissolution and transport of metals is particularly observed for areas draining through shale substrates.

The reduction in water quality or discolouration is generally restricted to areas close to the groundwater discharge or upwelling zones, and does not extend over large areas as the iron hydroxides precipitate and settle out near to the discharge area.

Stream Bed And Bank Erosion

Bed and bank erosion due to subsidence can have significant effects on stability of streams. This is particularly noticed on the upstream section of a subsidence troughs as headcuts can be generated in a stream bed that can
migrate for hundreds of metres upstream if not managed appropriately. The degree of bed and bank erosion depends on topographic changes endured by the stream, as well as the

- clayey or sandy nature of the stream bed and banks,
- soil dispersivity,
- height of stream banks,
- degree of destabilisation of the banks
- orientation of the stream to the subsidence trough
- seasonal or storm water flow variability
- logs, crossings, culverts or other features that can divert or confine flows
- bedrock in the stream bed, and
- interaction of tensile stress zones with stream banks

The stream attempts to resurrect its original gradient and stability regime after a subsidence trough or troughs intersect it. The adjustment generally takes the initial response of cutting down the stream bed over chain pillars and filling in subsidence troughs. Relocation of flow paths within the channel can erode the stream banks, whilst the headcut gradually migrates upstream. Without bed and bank erosion protection, the stream can adjust to a new regime over many years, with significant extension of the bed width and elongation of the headcut.

Adverse Ecosystem Changes

Ecosystems dependent on the stability or seasonal variation of a particular surface water body can be significantly affected by subsidence, particularly in the worst case where the water body is drained dry. Other effects that may occur are:

- Loss of ecosystem interconnection and potential transfer / replenishment of species. This is particularly noticeable where seasonal fish migration is affected by drying up of stream sections due to mining induced water loss.
- Changes in water temperature, dissolved oxygen, turbidity, and algal growth. The alteration of water depth, recharge versus discharge and the residence time in a body of water can significantly affect the health and diversity of a surface water body.

Reduction in Cropping or Grazing Land Functionality and Productivity

The “functionality” of cropping or grazing land can be affected by development of subsidence troughs and soil cracking diverting or collecting surface runoff, along with associated erosion or saturation of ground with a greater potential for bogging machinery. Other aspects of functionality can be present such as modified cattle access to streams, submerged or modified creek crossings, cracking of dams, and surface water ponding.

Subsidence effects on the wheat cropping potential of subsided land are being studied for an ACARP research project in the Bowen Basin by the Centre For Mined Land Rehabilitation. Although the project is not completed, initial indications are that the crop is not significantly affected. Overseas research, however, has indicated that cropping productivity is adversely affected

Diversion of Stream Flow Lines and Ponding

Drainage patterns of streams may be affected by diversion of flow out of the original channel. This may occur where subsidence troughs are not parallel, or indeed, perpendicular to the stream channel, with the drainage taking a preferential path along the subsidence trough. In some cases, particularly where the remnant ground height over chain pillars between subsidence troughs is higher than the banks of the stream, the stream may be dammed at the chain pillar and diverted along the trough.

Ponding of water in subsidence troughs can occur in paddocks or it can cause deepening and widening of pools in a stream, which if not filled in with subsequent sediment movement can lead to alteration of riparian ecosystems and geomorphological stability of the area.
Breaching or Dewatering Farm Dams

Breaching and/or dewatering of farm dams is possible due to subsidence crack development either in the walls or floor of farm dams. This may cause either a gradual loss of water or short term “catastrophic” outflows which may be a danger to people, animals or vegetation communities downstream of the dam.

Subsidence of Lake Shorelines

Subsidence of lake shorelines has been noted in the Newcastle Coalfield, where significant beach stabilisation and rehabilitation have been required.

GROUNDWATER

Dewatering or Interconnection of Aquifers

Subsidence and associated bedrock cracking over longwall panels can result in either dewatering of shallow aquifers separated by underlying aquitards which can in turn drain to the mine to underlying aquifers, or both. Development of cracks in the overburden can also allow deeper aquifers under high head to rise up into shallower aquifers.

Groundwater supply bores or monitoring piezometers can be significantly affected by lateral shearing of the bedrock, essentially by cutting the bore/piezometer off, which may necessitate a new water supply bore to be drilled to re-establish the water supply.

Reduction of the groundwater table may be permanent, in the case where groundwater drains into the mine or some other discharge feature, or temporary if the water level falls due to groundwater filling the increased secondary porosity of the bedrock due to development of new cracks. Once the new voids are filled, and on the assumption that there is no outflow from the regional system, the groundwater table will resume its original level. This recovery depends on the rate of recharge, with slower recoveries during drought periods.

The effect of dewatering may be temporary, as has been observed in the plateau country in the Southern Coalfield, but in some cases, a temporary water supply is required to replace the lost groundwater supply.

Reduction in Spring Flow and/or Stream Baseflow Recharge

The location and flow rate of springs in hillsides can be affected by subsidence cracking. This can occur if a spring is located on a hill due to the presence of an underlying aquitard limiting vertical migration of recharge through overburden. If the aquitard is breached, the spring may dry up or have its discharge rate reduced, with an associated flow increase to springs further down the slope.

Groundwater baseflow to streams can also be affected, with either an increase or decrease in discharge rate to streams, as well as a shortened response time to recharge and discharge. Higher discharge flow rates may occur due to the enhanced permeability of the aquifer due to crack development, however the duration of flows may be shortened as the aquifer drains out faster.

Emergence of the Groundwater Table

Groundwater in shallow aquifers can “daylight” if the depth of subsidence exceeds pre mining standing water levels over mined out panels. In this case, the ground level falls to a lower RL within the subsidence trough, whereas the water table remains at the same pre mining RL as its level is determined by catchment wide, rather than localised factors.

Flows into mine

Significant adverse health and safety issues can develop in mines where subsidence goaf and surface cracking intersect, thereby allowing aquifers in the overburden, streams flowing over the subsidence area or overlying lakes, dams or ponds to drain directly and rapidly into an underground mine.
This can generate a significant safety hazard to workers in the mine if catastrophic inflows occur whilst the mine is occupied, whilst ongoing inflows can cause a significant requirement to pump out excess water from the mine.

**Interference effects from adjoining mines**

Temporary or long-term depression of groundwater tables over subsided longwall panels can interact with adjoining open pit or underground leaseholders effects on the regional groundwater system, or alternately, adjoining operations can reduce standing water levels in a longwall lease.

**Seam or Overburden Gas Discharge to the Atmosphere**

Hydrocarbon gas comprising predominantly methane discharging from recently mined coal seams, or fractured overburden can vent to the atmosphere via interconnecting fractures, and/or through piezometers or wells within the mine subsidence fractured zone. This discharge can significantly disrupt water levels within the piezometer/well and cause temporary dewatering as it is “airlifted”.

This effect lasts as long as it be for the gas to vent off, which can take a few weeks to months.

**MANAGEMENT AND REHABILITATION OF ADVERSE CHANGES TO SURFACE WATER AND GROUNDWATER SYSTEMS**

The interrelationship of environmental issues described can, in some cases, be managed or rehabilitated by the same method. For instance, limiting water loss from a stream and reducing inflows to a mine close to surface can be achieved by sealing a stream bed and banks over the subsided section. This has been achieved to date primarily by either cement injection grouting of bedrock under a stream or rock pool or by sealing the channel with a compacted clay blanket buried beneath the sediment scour depth.

Reducing water losses from the stream or pool also has the advantage of enabling re-establishment of water dependent ecosystems and improving water quality in the affected area. Cement or clay based methods used to reduce stream flow losses do not, however, address stream bed or bank destabilisation and erosion, ponding of water or emergence of shallow groundwater.

Stream flow or water depth losses can also be rehabilitated by introducing compensatory inflow, such as treated town water, mine water or other inputs with acceptable water quality.

Rehabilitating bed and bank erosion may require a range of earthworks and sound erosion protection techniques that are designed to achieve specific outcomes. Each situation requires a specific remedy, or combination of rehabilitation approaches.

Management of headward bed erosion due to a stream cutting back or filling induced undulations over chain pillars and subsidence troughs generally involves installation of buried, keyed in rock weirs at the point of headward erosion. A stream cutting down a channel over a chain pillar will deposit sediment in the subsidence troughs as it attempts to re-establish its original gradient, however significant downstream discharge of suspended sediment may occur in the process. These sediments may need to be restricted from moving by additional sediment collection weirs. In perennial streams, weirs may also need to account for fish passage.

Stream bank erosion can generally be controlled by regrading the streambank and revegetating the area after installing fences to keep out cattle. Cattle can have a significant deleterious effect on bed and bank stability, with or without the added effect of subsidence.

Water quality can be improved by dilution with a suitable rate and quality of input supply, however iron hydroxides formed through upwelling of ferrous iron laden groundwater cannot be stopped unless the groundwater recharge/discharge relationship can be managed. Generally, the iron hydroxide concentration reduces over time, as the bedrock cracks become oxidised over months rather than weeks.

Adverse effects on cropping or grazing land through soil cracking and subsidence can be rehabilitated by deep ripping the cracks, however diversion of flow lines or water ponding may require additional drainage works, with the works planned and implemented in consultation with the DLWC.
Leaking or breached farm dams can be rehabilitated by resealing the dam wall and base and or building up the wall to an appropriate level.

Little can be done to rehabilitate dewatered aquifers or the interconnection of aquifers, as the subsidence crack development extends over the entire subsidence area, and grouting the overburden is generally not economically or logistically viable. Adverse effects on spring flow or groundwater stream recharge is also difficult to rehabilitate, except to provide suitable compensatory inflows.

Emergence of the groundwater table is also difficult to rehabilitate, unless the area is backfilled and rehabilitated to its original height, with this option having its own set of difficulties.

Groundwater flows into a mine can be significant, dangerous and costly. Inflows may be high for an initial period depending on the connectivity of the aquifer, its transmissivity, recharge and other factors, however over time, flows generally decrease as the aquifer is dewatered.

REFERENCES

<table>
<thead>
<tr>
<th>Issue</th>
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<td><strong>SURFACE WATER</strong></td>
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<td>Loss of Surface Water Flow / Water Depth</td>
<td>Cement, cement / bentonite grouting</td>
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<tr>
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<td>clay seal sediments below scour depth</td>
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<td>Fill in cracks with sediment / clay</td>
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<td>Reduced Surface Water Supply</td>
<td>Provide compensatory input by increasing upstream flows, trucking in water or</td>
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<td>Water treatment</td>
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<td>Re-establish flows to their original state</td>
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<td>Stream Bed and Bank Erosion</td>
<td>Battering back banks, installing headward erosion control weirs, and protection</td>
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<td>Fence off affected area and establish riparian vegetation</td>
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<td>Exclude cattle</td>
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<td>Reduced Land Functionality or Productivity</td>
<td>Deep rip cracks, fill in subsided areas,</td>
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<td>Diversion of Stream Flow Lines or Ponding</td>
<td>Install levees, or excavate drainage paths for collected water</td>
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<td>Reseal dam wall or floor with clay, and reform wall in extreme cases to original</td>
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<td>Lake Shore Subsidence</td>
<td>Rehabilitation the lake shore as appropriate</td>
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<td><strong>GROUNDWATER</strong></td>
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<td>Dewatering or Interconnection of Aquifers</td>
<td>Not much can be done on a mine scale, however isolated areas could be grouted at</td>
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<td>very high cost</td>
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<td>and logistically difficult)</td>
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<td>Emergence of Groundwater Table</td>
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<td>Establish a groundwater extraction programme</td>
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<td>Flows into a Mine</td>
<td>Seal the overburden by grouting, or divert surface water from over the mine</td>
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<tr>
<td>Interference Effects From Adjoining Mines</td>
<td>No cost effective options used to date</td>
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<tr>
<td>Seam Gas Discharge to the Atmosphere</td>
<td>Plug wells and piezometers, or wait for gas flow to abate</td>
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INVESTIGATIONS INTO PREMATURE ROCK BOLT FAILURES IN THE AUSTRALIAN COAL MINING INDUSTRY

Bruce Hebblewhite ¹, Mike Fabjanczyk ² and Peter Gray ³

ABSTRACT: An ACARP project was initiated in 1999 to address the observed phenomenon of premature failure of rock bolts in a number of Australian coal mines, and with a particular focus on the problem of Stress Corrosion Cracking (SCC) in rock bolts. This paper briefly outlines the findings of this study.

INTRODUCTION

SCC is a progressive fracture mechanism which can occur in different metals. It is not a new phenomenon and has been found in Bronze age swords, and in brass ammunition used in India in the last century. However, the effect of SCC on rock bolt failures has only been recognised in recent years.

STRESS CORROSION CRACKING

SCC occurs by the slow progressive growth of stress corrosion cracks under the joint action of stress and a corrosive environment affecting susceptible alloys. Eventually one of the cracks will reach a critical length at which the remaining section can no longer carry the load and final instantaneous overload failure then occurs. The stress corrosion cracks act as very sharp stress concentrators so that even small stress corrosion cracks can cause bolts to fail at below their design strength. An example of SCC cracks in a rock bolt is shown in Figure 1.

FIG. 1 - SCC Cracking in a Rock Bolt

SCC is not restricted to old bolts – some recently installed bolts have been found to fail under SCC conditions. It appears to be more prevalent where:

- Clay bands are intersected by the bolts
- Thick coal roof sections are present

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• High tensile steel bolts are used
• There is some groundwater present in the strata
• There is limited shearing within the strata inducing bending in the bolts
• Bacterial ‘bug’ corrosion of steel underground may be active, promoting corrosion of existing flaws within the steel.

Examples of SCC brittle failures in rock bolts are shown in Figure 2.

Results of the Study

The current project has surveyed the industry and found that the problem, although not identified in all mines, is more than just an isolated problem. It is not confined to just one coal seam where particular corrosive conditions apply or to all areas of that seam. It is not confined to one manufacturer or steel supplier.

The data collected during this project does indicate that there are still significant problems in defining the scale of prematurely fractured bolts in coal mines. The causes of the uncertainty can be summarised as:

• Lack of a consistent, or formal reporting system for the identification of incidents of broken bolts.
• The results of any survey of premature bolt failures is limited because only bolts that fail outside the encapsulated length of the bolt and are free to fall out or significantly displace out of the roof are identified.
• To date the only means of identifying bolts that fail in the encapsulated section is through inspection of fall cavities.
• Even in areas of high broken bolt frequency no broken bolts were identified in high deformation roofs. With the exception of failure of the collar of the bolt – bolts may be locked into the roof by shear action.
• Identification of the steel source is not possible without recourse to metallurgical analysis even for bolts with similar profiles.

There were at least four main types of premature bolt failure identified as part of this study. These were:

a. traditional stress corrosion cracking failures caused by corrosion cracks or pits propagating until ultimate brittle failure occurs as shown in Figure 1;
b. brittle failures within the encapsulated section of the bolt without any visible corrosion see bottom bolt in Figure 2;
c. existing cracks or pits in the steel which propagate and are the trigger mechanism for ultimate failure as shown in Figure 3;
d. failures within the threaded section of the bolt typically across the root diameter of the thread as shown in the right hand bolt of Figure 4).

FIG. 3 - Corrosion Pitting on the bolt surface – may also be an incipient failure point for rock bolts.
Metallurgical Characteristics of the Failed Bolts

Most failed bolts that were recovered from the mines were substantially straight, see Figure 2, but this is considered to be simply caused by biased sampling because straight bolts are less likely to be locked into the roof. In fact, some shear loading mechanism is probably required to explain bolt failures, below the level of encapsulation, with very low tensile collar loads.

Fracture depth in bolts varied from <1mm to over 40% of the surface area of the bolt (as shown in Figure 5 below).
A series of metallurgical tests have been applied extensively to the sampled bolts to identify metallurgical characteristics of the failed bolts and the nature of the corrosion. These point to steels with a low fracture toughness as particularly susceptible to this type of problem, and related brittle fracture in the vicinity of the threaded section of the bolt (toughness is a measure of the energy required for fracture).

The latest developments of higher fracture toughness steels in Australia has produced variable fracture toughness results as measured by the Charpy test. This may be due in part to actual variability in the steels but also the inability of the Charpy test to adequately measure fracture toughness on quench and tempered or accelerated cooled steels. Alternative indicators of fracture toughness for rock bolts should be considered (e.g., bending tests or drop tests).

Anecdotal evidence and the latest use of higher fracture toughness steels by mines, indicates that higher fracture toughness steels are less susceptible to premature failure than steels with low fracture toughness. Tempcored steels, accelerated-cooled steels and micro-alloyed steels in particular, appear to reduce the incidence of premature failures. However, the level of fracture toughness required to prevent premature bolt failures requires further investigation. The tensile, elongation and charpy values for various rock bolt steels are shown in Figure 6 below.
In addition to identifying that the problem of premature bolt failure due to SCC is occurring in a number of mines in both NSW and Queensland, the current project has accessed a considerable database of similar problems from the UK coal industry over the past ten years or more. Furthermore, it appears that SCC is also surfacing as a significant problem in a number of Australian metalliferous mines. In such cases, the prevalent use of point anchored bolts results in an even more severe problem due to 100% loss of capability on failure. A research project has recently been initiated in Western Australia into this problem in the metalliferous sector and agreement has been reached for collaboration through future information exchange.

SUMMARY OF THE FINDINGS

The problem has now been identified in 9 different collieries throughout NSW and Queensland. Samples of broken bolts have been collected from at least 5 of these mines for detailed metallurgical testing. Because of the sporadic occurrence of premature bolt failure and the low level of record keeping within the mines, only a limited sample is available for the current database. At this stage, only bolts which have fallen to the ground are normally recognised as premature failures. To date there has been no method of identifying broken bolts which have failed within the encapsulated portion of the bolt or are restrained in the roof by strata shear.

Approximately 50 different bolts were analysed as part of this study. These included AVH, AXR, X, HPC, Threadbar, Wriggle and Tempcored bolts. The testing program included crack detection through magnetic particle inspection; spectrographic analysis for chemical composition; toughness and impact testing using the Charpy test; plus micro-structural analysis of bolt steels. Additionally, confidential data available for use in the overall database was obtained from mines where a further 50 bolts failed.

Arising from this work has come a clear indication that the steel toughness is a critical factor in this type of failure, with lower toughness values being more prone to SCC failure – at least in certain mine environments. The majority of the failed bolts examined had very low Charpy impact values (4–7 Joules). There is a definite need to develop a much simpler and cheaper test for toughness, to enable an improved ‘quality control’ process to be employed by the industry.

It is also apparent that the SCC problem is almost entirely confined to the high tensile strength bolts, as opposed to mild steel – by virtue of the metallurgy involved. It is noted that the failed bolts examined all contained 0.4% - 0.6% carbon, but they were of a number of different chemical types, including manganese steels, chrome steels and micro-alloyed steels.
A further aspect of the database work involved detailed underground investigation in at least one mine where the problem was widespread, in order to assess any features of the mine environment, or the age or type of bolts which were more prone to SCC failure. This confirmed that the presence of a mildly corrosive groundwater will encourage SCC to develop. The existence of clay or tuffaceous bands within the bolting horizon appears to be one of the contributors to an SCC-prone environment.

A further feature of the site database development was a small-scale investigation into bacterial corrosion – iron and sulphur eating bacteria present in the mine environment, which corrode steel. This investigation was additional to the original project objectives, and was only conducted at a very low level, on a restricted sample, but it did confirm that these bacteria were prevalent at least in the one mine investigated.

The database also identified a number of bolt failures occurring in the threaded region of the bolts – apparently due to simple brittle failure in tension. This was observed as particularly relevant to bolts used for hanging loads such as belt structure, monorails and the like. This type of failure was not part of the original project, but is considered a significant issue, which warrants further investigation.

NON-DESTRUCTIVE BOLT INTEGRITY TESTING

It was recognised from the outset that the database obtained from broken bolts found on the floor of mine roadways may only be a small part of the problem. The type of failures being identified could just as easily be occurring within the grouted horizon. This would leave partially or totally failed bolts grouted into the roof, providing the potential for an extremely hazardous situation of roof instability developing, with little or no warning to operators. It was recognised that there was a need for a non-destructive device to be available to the industry to be able to check bolt integrity on a routine basis.

Several potential devices, or prototype developments, were identified and investigated as part of the current project. These included developments from the USA, Sweden, UK, and Germany. The most promising of the techniques, that was also the furthest developed, and already suitable for routine use in coal mines (with respect to cost and approvals) was an ultrasonic device developed by Deutsche Montan Technologie (DMT) in Germany. Discussions were initiated with DMT to enable trialling of the device in Australia. The German results, to date, have been very promising with this device, at least for detecting total bolt failure, and identifying the horizon where such failure has occurred.

DMT agreed to work with the ACARP project team to conduct further evaluations for application in Australian conditions. The first part of this agreement was a small test program in the DMT laboratories of similar types of bolts to those used in Australia. This testing yielded promising results. A more complete laboratory and field testing program of the DMT device should be pursued as part of any future investigations.

CONCLUSIONS

The current project has effectively characterised the problem and focused on some of the key metallurgical and environmental conditions under which the problem has to date been detected. A real concern is that the database, to date, only consists of bolts that have failed below the grouted horizon and dropped out onto the floor. Because of the nature of the problem, there is a very real likelihood that under the right conditions - some bolts are failing within the grouted horizon but remaining in the roof (at least two examples of broken bolts exposed in falls has indicated premature failure of bolts within the encapsulated section of the bolt). These give the appearance of a competent bolt, but one which may in fact be failed, in situ, and be effectively only offering a greatly reduced bolt length. The safety implications of this problem are considerable, and warrant priority future investigation.

The critical issues that should be addressed in the future, by the industry, include the following:

- Comprehensive field evaluation of the DMT developed ultrasonic NDT bolt testing device, to both prove the device for routine industry use, and further quantify the extent of the in situ bolt integrity problem.
- Further development of metallurgical and corrosion surface test procedures and database expansion – in particular, to develop and test a simplified steel 'toughness' test.
- Investigation and documentation of the properties of new steel products in the marketplace.
- Investigation of the extent of potential bacterial ‘bug’ corrosion of bolt steel and possible remedial actions.
• Documentation of the extent of brittle failure of bolts in threaded sections, especially with regard to bolts used for hanging structure, such as monorails.
• Provision to the industry of guidelines for minimizing SCC problems, including bolt and steel traceability.
• Introduction of an Australian standard for rock bolts. In particular, a minimum toughness level should be specified. This Standard should be based on the results of the key recommendations of this report, and include input from steel manufacturers, bolt producers and bolt users.

ACKNOWLEDGEMENTS

Support from the Australian Coal Association is gratefully acknowledged. Invaluable assistance was also provided by the staff at the various collieries from where samples were collected and surveys were undertaken and their support is also gratefully acknowledged.

REFERENCES

Heyes, P; 1999, “An overview on metallurgical work on rockbolt steel”; Health & Safety Laboratory, Research Project Reference Number: JS4000211; Sheffield, U.K.
LABORATORY TESTING OF ROCK BOLT STRESS CORROSION CRACKING

Erwin Gamboa and Andrej Atrens

ABSTRACT: The incidence of Stress Corrosion Cracking (SCC) in rock bolts has not been quantified and its magnitude has not been addressed. A laboratory test has been achieved that causes a tensile sample to fail in a manner similar to the failure mode observed from service failures, namely slow SCC followed by fast brittle fracture. The laboratory tests involve subjecting a tensile sample to a linearly increasing stress at a slowly applied stress rate whilst the specimen is exposed to a dilute sulphate solution of pH 2.1. Detailed fractography of SCC fracture features from the LT has shown that these fracture surfaces have the same features as fracture surfaces of service failures. An SCC velocity can be calculated from these tests. This SCC velocity can be used to evaluate the benefit provided by a material with a higher fracture toughness. The SCC velocity measured from the laboratory tests indicates that the SCC lifetime is increased only marginally by the use of a rock bolt materials with a higher fracture toughness.

Laboratory tests are being used to evaluate the threshold stress for rock bolts of various metallurgies and the environments causing stress corrosion cracking.

A hydrogen embrittlement mechanism for the SCC is indicated by the particular restricted range of conditions for which SCC occurs in the laboratory. In particular, SCC only occurs in the laboratory for the restricted range of environmental conditions corresponding to acid conditions at the open circuit potential (pH of 2.1 or more acid) or at negative applied electrochemical potentials corresponding to copious hydrogen evolution at the steel surface. This is consistent with reports from the USA indicating rock bolt failure due to the presence of H₂S in the mine atmosphere. Similarly, this failure mechanism is consistent with bacterial corrosion of the rock bolt surface during service producing acid conditions leading to SCC.

Water chemistry analyses carried out for a number of Australian mines (including one coal mine) visited during the 2002 suggest that SCC in a coal mine would be caused by bacterial corrosion locally decreasing the mine water pH down to a pH of 2.1.

INTRODUCTION

Any failure of a rock bolt is a potential concern. Failures of rock bolts have been reported at a number of Australian mines. The incidence of in rock bolts has not been quantified and its magnitude is not fully understood.

Atrens and Wang (1995) provide a review of SCC which may occur whenever a stressed steel is in the presence of an aggressive environment. The stress corrosion cracks initiate and grow slowly. During this phase which may last months or years there may be no indication of any danger. Fast fracture occurs when the stress corrosion crack reaches a critical length, as determined by the applied stress and the fracture toughness of the steel. Reports indicate that the critical crack length can be of the order of only a few millimetres for rock bolts. The fast fracture is sudden and catastrophic.

Crosky et al (2002) provided a recent review. They analysed approximately 50 different rock bolts, including “AVH, AXR, X, HPC, Threadbar, Wriggle and Tempcored bolts”. All the failed bolts examined “contained 0.4% - 0.6 % carbon, but were of a number of different chemical types, including manganese steels, chrome steels and micro-alloyed steels”.

1 University of Queensland
2 University of Queensland
The fracture surfaces of rock bolts (composed of Steel A) which failed in service due to SCC have been described by Gamboa and Atrens (2002a). Subsequent laboratory research by Gamboa and Atrens (2002b) has shown that service failures can be duplicated in the laboratory using the Linearly Increasing Stress Test (LIST). The LIST test involves subjecting a tensile sample exposed to a sulphate pH 2.1 solution to a linearly increasing stress until fracture, with an applied stressing rate of 0.019 MPa/s. The indications were that this test could provide a good method to reproduce service SCC in the laboratory.

Laboratory tests were carried out to study the SCC of rock bolts, to measure the SCC threshold stress for different steel metallurgies and explore the influence of galvanising on rock bolt SCC.

EXPERIMENTAL DETAILS

Materials and Specimens

LIST samples were tensile samples with a gauge section of 20 mm x 3.5 x 2.5 mm. LIST samples were machined from commercial rock bolts for four different steel metallurgies from actual rock bolt samples: Steel A, Steel B, Steel C and Steel D. Table 1 gives typical values of the chemical compositions of these steels. Table 2 gives typical values for ASTM grain size (D) and mechanical properties.

Table 1 Typical Chemical Compositions of the Rock Bolt Steels

<table>
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<tr>
<th>Steel</th>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Ni</th>
<th>Cr</th>
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<tr>
<td>A</td>
<td>0.54</td>
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<td>B</td>
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<td>0.013</td>
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Table 2 Typical ASTM Grain Size (D) and Mechanical Properties of the Rock Bolt Steels

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<tr>
<th>Steel</th>
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<th>YS MPa</th>
<th>UTS MPa</th>
<th>E</th>
<th>RoA %</th>
<th>YS/UTS</th>
<th>CVN</th>
<th>CVN</th>
<th>CVN</th>
<th>CVN(Mean)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>75</td>
<td>622</td>
<td>954</td>
<td>18</td>
<td>38</td>
<td>0.65</td>
<td>6</td>
<td>6</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>B</td>
<td>65</td>
<td>635</td>
<td>873</td>
<td>22</td>
<td>50</td>
<td>0.73</td>
<td>17</td>
<td>23</td>
<td>14</td>
<td>18</td>
</tr>
<tr>
<td>C</td>
<td>43</td>
<td>689</td>
<td>838</td>
<td>21</td>
<td>52</td>
<td>0.82</td>
<td>33</td>
<td>26</td>
<td>29</td>
<td>29</td>
</tr>
<tr>
<td>D</td>
<td>745</td>
<td>890</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Steel A contains 0.54% C in table 1 and 1.63% Mn in table 1. As the steel is made from scrap steel, it contains residual elements such as Cr, Si, Ni, Cu & Mo. A computer program is used to modify the Mn content of the melt according to the contents of the Cr, Si, Ni, Cu & Mo residuals, so that the steel has a 650 MPa minimum Yield Strength with no heat treatment. The steel has a nearly fully eutectoid microstructure.

Steels B and C are micro-alloyed. They contain the carbide former V, which is used to control the austenite grain size during hot working, and are subjected to careful control of the hot working sequence so that they produce a fine grained microstructure of the required mechanical properties. They have lower carbon contents and higher values of Charpy notch toughness (CVN values in Table 2).

Steel D has a composition similar to that of the Steel A. Strengthening is produced by ~ 10% cold work.

The SCC tendency of galvanised rock bolts was evaluated using LIST samples of Steel A, that were hot dipped galvanised to have a 100 μm zinc layer. Galvanised specimens were subjected to a standard LIST test in the sulphate pH 2.1 solution.

SCC Experiments

Figure 1 shows a schematic of the LIST apparatus from Atrens et al (1993). The apparatus is based on the principle of a lever beam. One side of the lever beam is connected to the specimen whilst the other side has a mass of 14 kg. Movement of the mass away from the fulcrum increases the load on the specimen. The applied engineering stress is calculated from the position of the mass at any time and the original cross section of the specimen.
The standard LIST experimental procedure was as follows. A specimen was inserted into the environmental cell and connected to the loading arms. The environmental cell was filled with the desired electrolyte. The LIST specimens had a free corrosion potential of -450 mV(SHE) in the standard sulphate pH 2.1 solution. The travelling mass was set in motion. Movement of the mass steadily transferred an increasing load onto the LIST sample. The mass was kept travelling until the sample fractured. All samples were loaded at 0.019 MPa/s, as the prior work by Gamboa and Atrens (2002a 2002b) indicated that the LIST test at this rate in the standard sulphate solution reproduced in the laboratory the same type of SCC fracture as observed in service. After the LIST test, the fracture surfaces were examined by Scanning Electron Microscopy (SEM).

The threshold stress was determined using a modified LIST test. The mass was stopped at a predetermined position and the specimen was held at a constant stress for 3 days in the standard sulphate pH 2.1 solution. If the sample did not fail by the end of the three day period, it was removed from the LIST apparatus, cooled to -197°C by immersion in liquid nitrogen, quickly withdrawn clamped in a vice and struck with a hammer, breaking the sample into two pieces at the thinnest part of the test section. The fracture surface was observed with SEM to determine the failure mode of the sample, and in particular whether a stress corrosion crack had formed. If no SCC crack was found, it indicated that the stress was below the threshold stress for SCC.
Test Solutions

All solutions were made using reagent grade chemicals and distilled water. Two standard solutions were used: chloride based and sulphate based. These might be characteristic of two different chemistries that might be found in underground water samples at mine sites. The following provide the details of these two standard solutions:

- Sulphate pH 2.1 solution. This contained 300 ppm sulphate, 100 ppm chloride and 100 ppm carbonate. This solution was made up as follows: 1.6543 g H$_2$SO$_4$, 0.3285 g NaCl and 0.5959 g Na$_2$CO$_3$ was dissolved in distilled water to make up 1000 mL of solution. The pH of this solution was measured to be 2.1. This solution is designated as "Sulphate pH 2.1".

- Chloride pH 1.8 solution. This contained 1400 ppm chloride, 300 ppm sulphate and 100 ppm carbonate. This solution was made up as follows: 1.6543 g H$_2$SO$_4$, 4.6056 g NaCl and 0.5959 g Na$_2$CO$_3$ was dissolved in distilled water to make up 1000 mL of solution. The pH of this solution was measured to be 1.8. This solution is designated as "Chloride pH 1.8".

Preparation of Fracture Surfaces

Macro-photographs were typically used to record the macroscopic appearance of the fracture surface of the rock bolt. Then the rock bolts were cut 10 mm below the fracture surface, cleaned using a 5% EDTA (ethylene diamine tetra-acidic acid disodium salt) solution, mounted on an aluminium stub, carbon coated and examined using scanning electron microscopy (SEM). Fracture surfaces of LIST specimens were prepared similarly. The LIST specimens were cut 10 mm below the fracture surface, cleaned using a 5% EDTA solution, mounted on an aluminium stub, carbon coated and examined using SEM.

RESULTS

A detailed comparison was carried out between LIST tests of Steel A exposed to the Sulphate pH 2.1 solution and Steel A rock bolts failed in service. There were the same fracture modes in both the service and laboratory samples. The fractures involved a small SCC region followed by a large fast fracture (FF) region. Figure 2 provides a typical overview for a service fracture. The images of Figure 2 represent optical images. The SCC region is easily identified by its darker colour due to the presence of corrosion products on the metal surface in the SCC region. This is due to the fact that the stress corrosion cracks grew slowly, allowing a period of time for corrosion to occur. In contrast, the overload region occurred essentially instantaneously, so that the surface was bright and shiny as an un-oxidized steel surface. The tear lines radiated away from the fracture origin. These tear lines facilitated the identification of the SCC feature that initiated the final fast fracture event. It was particularly noteworthy that the service stress corrosion cracks often initiated in association with the ribs on the surface of the rock bolt as illustrated in Fig. 2(b).
A direct comparison of a service fracture and a LIST fracture is provided in Fig. 3. Again, there was the SCC region and the brittle fast fracture (FF) of the overload region. These regions are identified in the schematics of Fig.s 3(b) and (d). It is worth noting that the fracture surfaces are macroscopically brittle with little indications of any macroscopic ductility.
FIG. 3(a) - Service Failure

FIG. 3(b) - Schematic of FIG. 3(a)

FIG. 3(c) - LIST Sample with SCC
Within the SCC region there were three different and distinct fracture morphologies: Tearing Topography Surface (TTS), Corrugated Irregular Slopes (CIS), and Micro Void Coalescence (MVC). These are illustrated in Figures 4-6.

FIG. 4(a) - TTS observed within a rock bolt SCC region
FIG. 4(b) - TTS observed within a rock bolt SCC region

FIG. 4(c) - TTS observed within a LIST SCC region
Typical TTS morphologies for rock bolts are presented in Fig. 4(a) & (b) to allow comparison of typical TTS morphologies from LIST samples in Figures 4(c) & (d). The tearing topography surface was characterised by a flat convoluted surface with tiny ridges apparently oriented randomly. TTS typically occurred close to the free surface and consequently was associated with the early stages of SCC. TTS has previously been described by Toribio and Vasseur (1997) as “a characteristic microscopic fracture mode with a kind of ductile tearing appearance, a certain degree of plasticity and a very closely spaced nucleation”.

Typical CIS morphologies for rock bolts are presented in Figures 5(a) & (b) to allow comparison of typical morphologies from LIST samples in Figures 5(c) & (d). The CIS surface was characterised by flat plateaus separated by corrugated slopes.
FIG. 5(b) - CIS observed within a rock bolt SCC region

FIG. 5(c) - CIS observed within a LIST SCC region

FIG. 5(d) - CIS observed within a LIST SCC region
Typical morphologies for the MVC-FF transition for rock bolts are presented in Figures 6(a) & (b) to allow comparison of typical morphologies from LIST samples in Figures 6(c) & (d). The MVC within the SCC region was significantly flatter than the dimple rupture observed in the overload region of tensile samples without SCC.

**FIG. 6(a)** - SCC(MVC)-FF transition observed within a rock bolt fracture surface

**FIG. 6(b)** - SCC(MVC)-FF transition observed within a LIST fracture surface

**FIG. 6(c)** - SCC(MVC)-FF transition observed within a LIST fracture surface
Figure 6(d) SCC(MVC)-FF transition observed within a LIST fracture surface

Typical brittle fast fracture (FF) morphologies for rock bolts are presented in Figures 7(a) & (b) to allow comparison of typical FF morphologies from LIST samples in Figures 7(c) & (d). The FF morphology was typical of cleavage fracture.

IG. 7(a) - FF (cleavage) observed within a rock bolt fracture surface
FIG. 7(b) - FF (cleavage) observed within a rock bolt fracture surface

FIG. 7(c) - FF (cleavage) observed within a LIST fracture surface

FIG. 7(d) - FF (cleavage) observed within a LIST fracture surface
Figure 8 proves an overview of the SCC region of a LIST fracture surface, with the schematic of Figure 8(b) identifying the various fracture micro-mechanisms.

FIG. 8(a) - Mosaic of the SCC region of a LIST fracture surface
Environments Causing SCC

The results of the SCC tests in the various experiments are summarised in Table 3 where S indicates a sulphate solution and C indicates a chloride solution. In the first tests only one environmental factor was changed (either the pH, the concentration or the corrosion potential), whereas in subsequent tests two factors were modified in order to study interactions, for example the pH and the corrosion potential. The data indicates that SCC was controlled by the combination of the applied potential and the pH. Furthermore, the data indicates that the solution concentration was not an important issue.
Table 3 Results of the standard LIST test for Steel A in various environments

<table>
<thead>
<tr>
<th>Sample</th>
<th>Environment</th>
<th>SCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST24</td>
<td>S pH 2.1</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST41</td>
<td>S pH 2.1 and CO₂</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 38</td>
<td>S pH 2.1 and coal, aerated</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 44</td>
<td>S pH 2.0, x10 conc</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 43</td>
<td>S pH 2.0, x100 conc</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 42</td>
<td>S pH 2.0, x1000 conc</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 28</td>
<td>S pH 4.2</td>
<td>No</td>
</tr>
<tr>
<td>LIST 29</td>
<td>S pH 6.3</td>
<td>No</td>
</tr>
<tr>
<td>LIST 34</td>
<td>S pH 9.4</td>
<td>No</td>
</tr>
<tr>
<td>LIST 47</td>
<td>S pH 2.1 Ecorr</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 48</td>
<td>S pH 2.1 Ecorr+100 mV</td>
<td>No</td>
</tr>
<tr>
<td>LIST 50</td>
<td>S pH 2.1 Ecorr+150 mV</td>
<td>No</td>
</tr>
<tr>
<td>LIST 49</td>
<td>S pH 2.1 Ecorr-100 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 51</td>
<td>S pH 2.1 Ecorr-300 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 52</td>
<td>S pH 6.2 Ecorr</td>
<td>No</td>
</tr>
<tr>
<td>LIST 53</td>
<td>S pH 7.27 Ecorr-300 mV</td>
<td>No</td>
</tr>
<tr>
<td>LIST 54</td>
<td>S pH 7.46 Ecorr-570 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 45</td>
<td>S pH 6.16, x100 conc, Ecorr</td>
<td>No</td>
</tr>
<tr>
<td>LIST 64</td>
<td>S pH 6.6, x100 conc, Ecorr-300 mV</td>
<td>No</td>
</tr>
<tr>
<td>LIST 66</td>
<td>S pH 6.6, x100 conc, Ecorr-500 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 85</td>
<td>S pH 1.2, Ecorr-150 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 87</td>
<td>S pH 1.2, Ecorr+100 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 88</td>
<td>S pH 11.8, Ecorr-850 mV</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 89</td>
<td>S pH 11.8, Ecorr-500 mV</td>
<td>No</td>
</tr>
<tr>
<td>LIST 26</td>
<td>C pH 1.8</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 39</td>
<td>C pH 1.8 and CO₂</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 40</td>
<td>C pH 1.8 and coal, aerated</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 46</td>
<td>C pH 1.8, x100 conc</td>
<td>Yes</td>
</tr>
<tr>
<td>LIST 31</td>
<td>C pH 3.1</td>
<td>No</td>
</tr>
<tr>
<td>LIST 37</td>
<td>C pH 7.7</td>
<td>No</td>
</tr>
<tr>
<td>LIST 36</td>
<td>C pH 10.7</td>
<td>No</td>
</tr>
</tbody>
</table>

The data has been displayed on an E-pH diagram in Figure 9 allowing identification of the conditions leading to SCC for LIST testing of Steel A samples. Sulphate solutions are represented by circles and chloride solutions are represented by squares. A full symbol indicates fracture by SCC, whereas an empty circle or square means that no SCC was detected. Symbols with a flag represent experiments performed with an applied potential.
FIG. 9 - LIST test results superimposed on an E-pH diagram. Full symbols indicated SCC, open symbols indicated ductile failure in the LIST test. Circles represented the sulphate solutions. Squares represented the chloride solutions. Symbols with flags indicated tests under potential control.

SCC only occurred in these laboratory tests for the restricted range of environmental conditions corresponding to acid conditions at the open circuit potential (pH ~ 2.1 or more acid) or at very negative applied electrochemical potentials corresponding to copious hydrogen evolution at the steel surface. This indicated that a hydrogen embrittlement mechanism was responsible for the SCC. This is consistent with reports from the USA indicating rock bolt failure due to the presence of $H_2S$ in the mine atmosphere. Similarly, this failure mechanism is consistent with bacterial corrosion on the rock bolt surface during service producing acid conditions leading to SCC.

**Determination of Threshold Stress**

The results of a testing program to determine the threshold stress for SCC to occur are summarised in Table 4. Some samples did not fail by SCC but displayed pits, which caused ductile overload. Samples that failed in this way have been classified as failing by pitting and a "P" designation has been given. An "NP" designation means that the sample did not have substantial pits, nor was the failure associated with pitting.

Steel A samples held at 770, 861 or 885 MPa did not fail in the LIST apparatus during the period of 3 days during which the load was held constant. Furthermore, these samples did not show any evidence of SCC when they were fractured at the temperature of liquid nitrogen. In contrast, the sample held at 922 MPa fractured after eight hours. The fracture surface showed a typical macroscopically brittle appearance typical of SCC causing a fast brittle fracture. Detailed SEM examination was consistent, showing a clear region of SCC followed by a FF region. Furthermore, surface corrosion damage of these Steel A samples was limited. There were wide shallow pits all over the gauge surface but there was no evidence of SCC.
**Table 4 Determination of the Threshold Stress [MPa] for SCC**

<table>
<thead>
<tr>
<th>Steel</th>
<th>Stress not causing SCC</th>
<th>Stress causing SCC</th>
<th>Threshold Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>770 (NP), 861 (NP)</td>
<td>922</td>
<td>900</td>
</tr>
<tr>
<td>B</td>
<td>700 (P after 1h), 800 (P* after 50h)</td>
<td>830</td>
<td>815</td>
</tr>
<tr>
<td>C</td>
<td>700 (P after 27h), 800 (P after 70h)</td>
<td>850</td>
<td>850</td>
</tr>
<tr>
<td>D</td>
<td>-</td>
<td>-</td>
<td>&gt; 960</td>
</tr>
</tbody>
</table>

*fractography indicated both pitting & SCC – test is being repeated.

“P” indicates pitting causing sufficient decrease in section to cause ductile overload failure after the specified period of load application.

“NP” indicates no pitting. For all tests of the Steel A samples, the samples did not fail during the 3 day exposure period to the specified stress.

The Steel D material showed ductile failure for the LIST test completed to fracture in the standard Sulphate pH2.1 solution.

The Steel B sample held at 830 MPa failed after 50 hours. The gauge surface displayed extensive surface corrosion and many stress corrosion cracks as shown in Figure 10. Samples held at lower stresses failed fairly rapidly by pitting. It is possible that SCC might have occurred at these lower stresses if failure had not occurred by pitting. A more accurate determination of the SCC threshold stress requires use of a different specimen with a substantially larger cross-section. A larger specimen would tolerate a larger pit size before the section was reduced sufficiently to cause ductile overload fracture. Pits typically decrease in propagation velocity as they grow in size. This means that for a specimen with a larger cross-section area, there would be much longer exposure before failure by pitting, providing more time for SCC initiation and growth.

**FIG. 10 - SCC along LIST gauge length of Steel B sample.**

The Steel C sample held at 850 MPa failed after 71 hours. The fracture surface displayed both ductile fracture (shown by the lip formed by plastic flow) and also displayed SCC features. Surface damage was extensive all over the gauge surface, with many pits and many secondary SCC illustrated in Figure 11.
The Steel \( D \) samples failed in a ductile manner, for the LIST test that proceeded until specimen fracture at an applied stress of 960 MPa. The fracture surface was typical of ductile fracture, Figure 12. The ductile failure surface illustrated in Figure 12(a) and the brittle fracture surface illustrated in Figure 3(a) may be compared.

Mine Water Chemistry

In order to compare the laboratory solutions to the mine waters, which represent the service conditions, samples were taken from various Australian underground mines. Table 5 presents the composition of the various mine water samples collected during 2002.

<table>
<thead>
<tr>
<th>Sample</th>
<th>pH</th>
<th>EC</th>
<th>Ca</th>
<th>K</th>
<th>Mg</th>
<th>Na</th>
<th>S</th>
<th>Cu</th>
<th>TDS</th>
<th>Cl</th>
<th>NO(_3)</th>
<th>Alkalinity</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC1</td>
<td>7.6</td>
<td>50.5</td>
<td>835</td>
<td>99</td>
<td>1188</td>
<td>14839</td>
<td>2090</td>
<td>0.015</td>
<td>47221</td>
<td>23600</td>
<td>2.12</td>
<td>312</td>
</tr>
<tr>
<td>EC2</td>
<td>7.2</td>
<td>48.8</td>
<td>809</td>
<td>98</td>
<td>1092</td>
<td>13382</td>
<td>1925</td>
<td>0.014</td>
<td>42161</td>
<td>22600</td>
<td>0.05</td>
<td>347</td>
</tr>
<tr>
<td>EC3</td>
<td>8.1</td>
<td>1.9</td>
<td>155</td>
<td>13</td>
<td>109</td>
<td>117</td>
<td>157</td>
<td>0.017</td>
<td>1508</td>
<td>1082</td>
<td>1.24</td>
<td>396</td>
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<td>EC4</td>
<td>8.3</td>
<td>1.4</td>
<td>91</td>
<td>17</td>
<td>108</td>
<td>96</td>
<td>132</td>
<td>0.018</td>
<td>1183</td>
<td>636</td>
<td>7.95</td>
<td>334</td>
</tr>
<tr>
<td>EC5</td>
<td>6.8</td>
<td>15.3</td>
<td>488</td>
<td>164</td>
<td>4100</td>
<td>831</td>
<td>5885</td>
<td>0.024</td>
<td>30436</td>
<td>83</td>
<td>1.45</td>
<td>211</td>
</tr>
<tr>
<td>EC6</td>
<td>7.1</td>
<td>15.4</td>
<td>489</td>
<td>165</td>
<td>4100</td>
<td>831</td>
<td>5760</td>
<td>0.029</td>
<td>31215</td>
<td>57</td>
<td>7.30</td>
<td>209</td>
</tr>
<tr>
<td>EC7</td>
<td>7.7</td>
<td>16.4</td>
<td>489</td>
<td>189</td>
<td>4420</td>
<td>938</td>
<td>6148</td>
<td>0.004</td>
<td>30480</td>
<td>84</td>
<td>3.09</td>
<td>176</td>
</tr>
<tr>
<td>EC8</td>
<td>7.9</td>
<td>4.4</td>
<td>113</td>
<td>9</td>
<td>165</td>
<td>750</td>
<td>179</td>
<td>0.018</td>
<td>3006</td>
<td>1736</td>
<td>0.99</td>
<td>968</td>
</tr>
<tr>
<td>EC9</td>
<td>7.7</td>
<td>4.1</td>
<td>423</td>
<td>9</td>
<td>56</td>
<td>546</td>
<td>464</td>
<td>0.035</td>
<td>3322</td>
<td>1434</td>
<td>0.14</td>
<td>182</td>
</tr>
<tr>
<td>Sulphate</td>
<td>2.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>388</td>
<td>540</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Chloride</td>
<td>1.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1942</td>
<td>540</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>50</td>
</tr>
</tbody>
</table>

All of these samples were taken from metalliferous mines, except for sample EC8 which was taken from a colliery. The composition of EC8 similar to that of the others samples. A comparison of the field samples and the standard sulphate pH 2.1 and the chloride pH 1.8 solutions indicated that the chemical composition of the laboratory solutions have similarities to those in the field, particularly to samples EC4, EC8 and EC9. Some of the field samples have much higher concentration of sodium, sulphate and chlorides than the laboratory standard solutions. EC1 and EC2 have more than 4 times the concentration of Na, S and Cl than the laboratory standard solutions. Even though field samples had higher concentrations, none of the mines sampled have reported problems of rock bolt SCC. This is consistent with the present results that show solution concentration was not an important factor causing SCC. The main difference between the field samples and the laboratory standard solutions is that the laboratory solutions have a much lower pH (1.8 and 2.1) compared to those from the field (6.8 to 8.3). It is hypothesised that if micro-organisms were introduced in those regions where the samples were taken the local pH conditions could drop into a region where SCC does occur.
Galvanised Samples

Galvanised samples subjected to the standard LIST test displayed ductile overload, failing at 856 MPa. The surface appeared cracked and the zinc layer had flaked. However, the core of the LIST sample (free of zinc) failed in a ductile manner. Ungalvanised samples failed at 950 MPa, indicating that the process of galvanising the LIST samples (i.e., the dipping into molten zinc) had the effect of lowering the strength of the steel. This was attributed to the heating of the steel to the temperature of the molten zinc causing a coarsening of the steel microstructure.

Figure 13 shows the fracture surface of the galvanised LIST sample and a magnified view of the fracture of the zinc layer. Fig. 13(a) shows the cracked zinc layer at the surface of the LIST specimen, whilst there was a ductile failure fracture surface for the steel. Fig. 13(b) presents a higher magnification view of the fracture of the surface zinc rich layer that had cracked and separated as the LIST sample deformed during the LIST test. The zinc layer did not only crack, but it also separated from the steel sample.

Fig. 13 Fracture surface of galvanised sample after LIST test
(a) overview. (b) detail of brittle fracture of zinc layer
Figure 14 summarises the controlled potential testing of galvanised samples. This showed that galvanised samples could be made to show SCC when a very negative potential was applied. This observation reinforced the identification of hydrogen embrittlement (HE) as the SCC mechanism.

**DISCUSSION**

A SCC velocity can be calculated from these tests using $v_{SCC} = \frac{d}{t}$, where $d$ = stress corrosion crack size at the onset of fast fracture, and $t$ = Laboratory Test (LT) duration. Typical values (for Steel A) are: $v_1 = 1.200 \times 10^6$ m/(50,400 s) = $2.3 \times 10^{-8}$ m/s; $v_2 = 500 \times 10^6$ m/(19,140 s) = $2.6 \times 10^{-8}$ m/s; to give an average value of $v_{av} = 2.5 \times 10^{-8}$ m/s. This crack velocity can be used to evaluate benefit provided by a material with a higher toughness. There is an increased lifetime between SCC onset and fast brittle fracture. If the fracture toughness is increased by a factor of two, the critical crack size in service is increased by a factor of $(two)^2 = 4$, e.g. from say $2 \times 10^{-3}$ m to $8 \times 10^{-3}$ m. The increased life time $t = \frac{d}{v} = 6 \times 10^3/ 2.5 \times 10^{-8} = 240,000$s = 2.7 days.

The threshold stress for Steel A rock bolt samples ~ 900 MPa, indicating that the loading causing SCC in service is due to a combination of the tensile load plus the bending load due to rock shear. This is consistent with the observation that rock bolts are typically bent after failure in service. This bending indicates a stress above the yield stress having been applied to cause the permanent deformation in bending, i.e. to cause permanent plastic deformation. The bending of the bolt has been attributed to shear in the rock strata. This leads to the issue of whether SCC could be prevented by a rock bolting strata design that prevented shear in the rock strata and thereby maintained the stress in the rock bolt below the threshold stress for SCC initiation.
CONCLUSIONS

- The incidence of SCC in rock bolts has not been quantified and its magnitude is not fully understood.
- A laboratory test has caused a tensile samples to fail in tension in a manner similar to the failure mode observed from service, namely slow SCC followed by fast brittle fracture. The laboratory tests involve subjecting a tensile sample to a linearly increasing stress at a slow applied stress rate whilst the specimen in exposed to a dilute sulphate solution of pH 2.1. Detailed fractography of SCC fracture features from the LT has shown that these fracture surfaces have the same features as fracture surfaces of service failures.
- The crack velocity measured from the laboratory tests indicates that the SCC lifetime is increased only marginally by the use of a rock bolt materials with a higher fracture toughness.
- The threshold stress for Steel A rock bolt samples is around 900 MPa, indicating that the loading causing SCC in service is due to a combination of the tensile load plus the bending load due to rock shear.
- The threshold stress for Steel C was in the order of 850 MPa and for Steel B, 830 MPa.
- Steel D bolts have experienced service failures, but Steel D rock bolt material did not show SCC in the laboratory test at the free corrosion potential in the standard sulphate solution, pH 2.1. This discrepancy indicates that (1) the susceptibility of Steel D should be explored at lower pH values, and (2) the influence of cold work on SCC should be studied.
- A hydrogen embrittlement mechanism for the SCC is indicated by the particular restricted range of conditions for which SCC occurs in the laboratory.
- In particular, SCC only occurs in the laboratory for the restricted range of environmental conditions corresponding to acid conditions at the open circuit potential (pH ~ 2.1 or more acid) or at very negative applied electrochemical potentials corresponding to copious hydrogen evolution at the steel surface. This is consistent with reports from the USA indicating rock bolt failure due to the presence of H₂S in the mine atmosphere. Similarly, this is consistent with bacterial corrosion on the rock bolt surface during service producing acid conditions leading to SCC.
- Water chemistry analyses has been carried out for a number of Australian mines including one coal mines. The water in all cases was neutral, with the pH ranging from 6.84 to 8.32.. The UQ laboratory test indicates that SCC would not occur in any of these neutral mine waters. This does indeed suggest that SCC in a coal mine would be caused by bacterial corrosion locally decreasing the mine water pH down to a pH ~ 2.1.
- Galvanised samples did not show SCC in the laboratory test at the free corrosion potential in the standard sulphate solution at pH 2.1, but SCC could be induced in the pH 2.1 solution at a more negative potential. A galvanised coating has a short life due to general corrosion in the pH 2.1 solution.
- Further research is required.

ACKNOWLEDGEMENTS

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REFERENCES

DOUBLE SHEAR TESTING OF BOLTS

Naj Aziz 1, Damian Pratt 1, and Richard Williams 1

ABSTRACT: Double shear testing was carried out on fully grouted and axially tensioned bolts installed in two different types of three piece concrete blocks. The purpose of the study was to examine the behaviour of reinforced bolts in shear under different axial loading conditions. A total of 22 bolts were tested using three common types of bolts used in Australia. The differentiating factor in bolt selection was the surface profile configuration, and the role of such configuration on the load transfer characteristic of cement/resin and bolt interactions. The influence of different tensional loads on the load transfer characteristics of bolts was also examined. The study showed that the medium strength and the axial tensional load influenced the level of shear load. Higher bolt profile configuration was least affected by the increased axial tensional load changes. There was no clear relationship between the vertical displacement at elastic yield point and each of bolt type, applied axial tensional loads and the medium strength. However, the shear loads were found to increase with increasing tensional loads and that the bolt profile configuration had influenced the shear load.

INTRODUCTION

The shear behaviour of reinforced joints and bedding planes has remained the subject of research for several decades particularly in geotechnical engineering. Bjurstrom (1974) was one of the early pioneers in the field. Based on his studies on granite specimens reinforced by fully grouted steel bolts, Bjurstrom reported that, the bolt failure characteristics were dependent upon the angle between the bolt and the joint planes, and that the bolt failure in tension occurred when the angle was less than 35°. He also suggested that the shear strength of such rock was dependent on, the shear resistance due to reinforcement effect, shear resistance due to the dowel effect and shear resistance due to the friction of joint of the host medium. Hass (1981) working on reinforced limestone specimens, with artificially cut joints, has concluded the orientation of rock bolts relative to the shear plane had the shear resistance offered by the bolt. Hass also stated that the shear strength of a bolted joint was found to be the sum of the bolt contribution and the friction resulting from the normal stress on the shear plane. Ferrero (1995) proposed a shear strength model for reinforced rock joints and suggested that the overall strength of the reinforced joint could be attributed to the combination of both the dowel effect and the incremental axial force due to the bar deformation. Ferrero’s analytical model was applicable to the bolts installed perpendicular to the joint plane in stratified bedding planes. Dight (1982) conducted an extensive study of the behaviour of a fully grouted bolt in shear, He developed a theoretical model with bolt failure in shear and was able to verify his model with experimental results.

There has been very little interest in examining the influence of the bolt surface profile to the joint or bedding interface reinforcement. Accordingly, a laboratory based study was undertaken to examine the effect of profile characteristics and configuration on load transfer mechanisms between rock/resin /bolt. Three well known bolt brands were selected for the investigation, and for the sake of unanimity they were given different designations.

EXPERIMENTAL PROCEDURE

Block Casting

Double-jointed concrete blocks were cast for each double shearing test. Two different strengths of concrete blocks were cast, 40MPa and 20MPa strengths to simulate two different strength rocks.

Once mixed the concrete was poured into greased wooden moulds measuring 600mm x 150mm x 150mm which were divided into three sections separated by two metal plates. A length of plastic conduit 24mm in diameter was set through the centre of the mould lengthways to create a hole for the bolt. The concrete was left for 24hrs to set and then removed from the moulds and placed in a water bath for a period of 30 days to cure. The plastic conduit

1 University of Wollongong
was removed from the centre of the blocks and the hole was reamed to 27 mm diameter, ready for the bolt installation.

1400 mm long bolt, threaded 100 mm on both ends was then fixed in the concrete specimen using Fosroc Chemfix PB1 Mix and Pour grout resin. Care was taken to install all the bolts in their respective concrete blocks with uniform profile /flash orientation. The bolted blocks were left for at least half an hour to allow the resin to cure before moving them for the place of storage. Most bolted specimens were left for days or up to few weeks before being tested.

The concrete/bolt assembly was then mounted in a steel frame shear box fabricated for this purpose. A base platform that fitted into the bottom ram of the Instron Universal Testing Machine, capacity 50 kN, was used to hold the shear box. Steel blocks about 55mm thick were placed beneath the two outer concrete blocks to allow for centre block vertical displacement when shearing load is applied. The two outer ends of the shear box were then clamped tightly with the base platform to avoid toppling of the blocks during shear loading. A predetermined tensile load was applied to the bolt prior to shear loading. This acted as a compressive/confining pressure to simulate different forces on the joints within the concrete. The three nominated tensions were 20KN, 50KN and 80KN. Axial tensioning of the bolt was accomplished by tightening simultaneously the nuts on both ends of the bolt. The applied axial loads were monitored by two hollow load cells mounted on the bolt on either side of the block. During testing, load-cell readings were taken every 20 kN at 0.04 sec /minute loading rate. The outer sections of the shear box remained fixed as the central block was pushed down.

A total of 28 bolts and concrete specimens were tested in the combinations shown in Table 1. Bolt type T3 was rather under-represented due to time constrain. Figure 1 shows a typical view of the assembled shear box unit in a testing machine and sketch of deformed bolt together with a number of post testing deformed bolts and sketch of deformed bolt together with a number of post test deformed bolts.

Table 1 – Experimental Schedule indicating the number of samples tests per bolt type

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>20MPa Concrete</th>
<th>40MPa Concrete</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20kN 50kN 80kN</td>
<td>20kN 50kN 80kN</td>
<td></td>
</tr>
<tr>
<td>AX</td>
<td>2 2 2</td>
<td>1 2 2</td>
<td>11</td>
</tr>
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<td>AXR</td>
<td>2 2 2</td>
<td>2 2 2</td>
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<tr>
<td>JAB</td>
<td>1 0 1</td>
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<tr>
<td>Total</td>
<td>5 4 5</td>
<td>4 5 4</td>
<td>28</td>
</tr>
</tbody>
</table>

FIG. 3 - Photograph of tested sample in Instron testing machine and sketch of deformed bolt together with a number of post testing deformed bolts.
RESULTS AND ANALYSIS

Shear Load and Shear displacement (deflection)

Figure 2 shows shear load and shear displacement (deflection) of bolts tested in both 20 and 40 MPa strength concrete and under different tensile loading conditions respectively. The shear load increased at the initial stage of loading until the elastic yield point was reached. The rate of loading began to decrease with increased shear displacement. Table 2 shows the shear load and shear displacements at elastic yield points for various cases. Figure 3 (a-f) shows the comparative shear load and vertical displacement profiles in both 20 and 40 MPa concrete medium respectively.

Table 2-yield point shear load values for different bolts under differing environment.

<table>
<thead>
<tr>
<th>Concrete strength (MPa)</th>
<th>Tensile load (kN)</th>
<th>Type 1 Shear Load at y/point (kN)</th>
<th>Type 1 Shear displace. at y/point (mm)</th>
<th>Type 2 Load at Y/point (kN)</th>
<th>Type 2 Shear displace. at y/point (mm)</th>
<th>Type 3 Load at Y/point (kN)</th>
<th>Type 3 Shear displace. at y/point (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>20</td>
<td>180</td>
<td>2.79</td>
<td>240</td>
<td>5.84</td>
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<td>240</td>
<td>4.86</td>
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<td>5.21</td>
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<td>80</td>
<td>240</td>
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<td>240</td>
<td>4.38</td>
<td>180</td>
<td>3.58</td>
</tr>
<tr>
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<td>20</td>
<td>100</td>
<td>4.89</td>
<td>160</td>
<td>8.86</td>
<td>80</td>
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<td>20</td>
<td>50</td>
<td>150</td>
<td>5.86</td>
<td>160</td>
<td>4.64</td>
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<td>5.84</td>
<td>160</td>
<td>4.69</td>
<td>180</td>
<td>4.61</td>
</tr>
</tbody>
</table>
FIG. 2 - Shear load and shear displacement of bolts tested in both 20 and 40 MPa strength concrete and under different tensile loading conditions respectively
The following can be noted from the shear load and deflection graphs:

1) The shear load of the bolt increased with increasing bolt tension for Bolt Type 1. There was almost no change in shear load in Bolt Type 2 and ironically there was a decline in shear load in Bolt Type 3. As shown in Table 1, only one test was made per test condition for Bolt Type 3, while two tests were made for other two bolts per given condition.

2) The strength of the medium has influenced the shear load level but not the trend. Shear load values for all bolts were generally less in 20 MPa concrete medium in comparison to the shear load values of bolts tested in 40 MPa concrete.

3) In general the shear displacement at elastic yield point was not consistent irrespective of the concrete type and the axial load. This was the same for all three bolt types.

4) Bolt Type 2 displayed constant shear load at all three levels of bolt tension loads in both 20 and 40 MPa concrete mediums. The consistency of shear loads at bolts elastic yield point was more pronounced that the other bolt types.
SHEAR LOAD AND LOAD CELL READINGS

Figure 4 shows a typical shear load versus load cells readings on tensile loads applied on a bolt in bolts installed in a 40 MPa concrete medium. Point A is known as the Limit of Maximum Frictional Bonding Strength (LMFBS) which indicates shear load values whereby the load cells values began to increase from the initial applied load. This level of shear load was significantly higher than the elastic yield point shown in Figures 2 and 3 respectively. The level of shear load increase was dependent on the initial axial tensile load on the bolt, concrete type and bolt profile pattern. Figure 5 (a-f) shows the different shear load and load cell readings for various bolts. The graph profiles were different for different bolts.

The following can be observed from these figures:

- The level of initial confining axial load applied to bolts had profound influence on the applied shear load at the Limit of Maximum Frictional Bonding Strength (LMFBS) between the bolt and resin. In the majority of cases the higher was the initial tensioning load, the greater was the shear load at LMFBS. However, there was exception to this kind of loading pattern as shown in Figure 5f. The above relationship occurred irrespective of bolt type and medium strength.
- The shear load values beyond was greater than the elastic point of the bolt.
- Back sloping of the Load Cell load/shear load graph prior to the failure of the frictional bonding strength was attributed to the crushing of the concrete blocks as indicated in Figures 5 d and f. Clearly the bolt appears to have pulled through the concrete as the shear load was increased. This phenomenon was more common in weaker strength medium such as in 20MPa concrete medium.
- Shear load vs Load Cell readings were more consistent in stronger concrete than that in weaker one. Excessive back sloping was evident in this bolt than the other bolts. Also the peak shear load at the limit of frictional bonding point was the lowest of all the bolts tested in 20 MPa concrete.
- As the holes made in the concrete blocks were not rifled, there is a reasonable chance that the relative movement between concert/resin/bolt could have been either between the bolt and resin or between any of the two elements and at worst a combined movement involving concrete/resin and bolt.
- Bolt Type 2 installed in the 40MPa concrete have comparatively greater shear load at LMFB point than the other two bolts.
- 20 MPa concrete was too weak for testing 22 mm core diameter bolts. Excessive negative sloping of the shear load and initial tensile loads of the bolts prior to reaching LMFB point values is a clear indication of the concrete block crushing at higher confining load as shown in Figure 6. In situ such situation may arise with excessive pretensioning of bolts installed in rock stratification with weaker layers sandwiched between two relatively competent layers.
FIG. 5 - Shear load and load cell readings for various bolts with initial tensile loading of 20, 50 and 80 kN
FIG. 6 - Post test end block view of bolt pulled through the concrete during shear loading

The experimental set up for this programme of study, using 150 mm$^2$ side blocks was found to be relatively small and inadequate to conduct tests of such large bolt size and load application magnitudes. Plans are currently underway to conduct similar studies in much larger blocks and in different types of rocks with rifled walled holes.

CONCLUSIONS

The double shear testing represent a useful method of assessing the bolt behaviour in multi stratified reinforcement. The technique demonstrated the role of bolt surface profiles with regard to load transfer characteristics at shear. The findings form this study were in agreement with others tested conducted with regard to the influence of bolt surface profiles on load transfer capacity of the bolts.

The technique highlighted the difficulties associated in small scale testing if concrete and testing in smooth wall holes instead of rifled holes drilled in-situ. The next phase of the study will include testing bolts in real rock samples and in much larger medium size.

REFERENCE

LONGWALL ROOF CONTROL BY CALCULATION OF THE SHIELD SUPPORT REQUIREMENTS

Ulrich Langosch, Ulrich Ruppel, Uwe Wyink

ABSTRACT: In the 1990s the German mining industry introduced a new generation of shield supports. The new design of support has a maximum load capacity of 10,000 kN, making these units as strong as the shields used in Australia and in the USA. Deutsche Montan technologie (DMT) took more than 3,100 underground observations in order to verify the roof fall frequency by statistical analysis. The results of this work have led to practical recommendations for roof control and the required shield support system on longwall faces.

The underground observations have been correlated to Rock Mass Classification, to stress calculations and to the angle between the direction of the fissures and the direction of longwall mining. The analysis work yielded the following two sets of results:

1. There is a critical distance between the canopy tip and the coal face Tip to Face (TF\textsubscript{crit}). The TF\textsubscript{crit} is predictable and relates to:
   - the thickness of the first roof layer and
   - its uniaxial compressive strength.

The face support should have a Tip to Face (TF) that is less than the TF\textsubscript{crit} in every underground longwall situation. Exceeding the TF\textsubscript{crit} can immediately result in a roof fall.

2. Using the obtained regression equation DMT is able to calculate the probability of the Roof Fall Frequency (RFF), which describes the roof fall sensitivity. When TF\textsubscript{crit} is exceeded the predicted FF relates to:
   - the Measured Support Resistance (MSR) of the shield support
   - the calculated vertical stress (p\textv)
   - the Fissure-Direction Index (FDI) which equals the angle between main fissure direction and direction of mining and
   - the distance by which TF\textsubscript{crit} is exceeded (ΔTF\textsubscript{crit}).

Armed with these results DMT is now able to predict the critical distance between the tip of the canopy and the coal face (TF\textsubscript{crit}), as well as the RFF, for all shield designs.

By applying the new calculation method it is now possible to compare alternative longwall layouts and different shield support types under pre-set geological conditions. Mining engineers on site are therefore in a position to make the necessary roof control preparations required to run the longwall operation to maximum efficiency. The results provide a useful basis for making practical recommendations and for selecting the most effective design of shield support.

Practical examples to demonstrate the R&D results and present the various methods now available for calculation and prediction in longwall roof control are presented.

INTRODUCTION

Prior to 1990 the German coal industry employed shield supports that were notable for their lightweight, compact design and high-strength steel construction (Figure 1). This meant that the 2-leg shields weighed in at a relatively light 10 to 15 tonnes. The maximum support load was between 5,000 and 6,000 kN. Most of these supports were fitted with jointed canopies with slide bars (extensible forepoles) to improve the contact between the canopy and

\[ \text{Deutsche Montan Technologie GmbH} \]
the often undulating roof beds. A process for evaluating the anticipated roof falls was already in place for this generation of shields as long ago as 1980 (Jacobi, 1981)

FIG. 1 - Shield support design in the 1980s

Some ten years ago the discussion in Germany began to focus intensively on the positive experience obtained in the USA and Australia with rigid, one-piece canopies and much higher maximum support loads that in some cases exceeded 10,000 kN. Around about the mid-1990s this resulted in a standardisation of shield support systems in the German coal industry. Nowadays the supports purchased by the industry are exclusively 2-leg shield units with one-piece roof canopies and support loads of up to 10,000 kN – these being subdivided into four categories for seam thickness (Figure 2). Most of the new standard shield designs also avoided using high-strength steels, which in some cases had increased the weight of the units to as much as 20 tonnes.

FIG. 2 - Shield support design since the mid-1990s

These standard shield supports are to be installed on all coal faces in Germany in the course of the next few years. However, as quite a few of the “old shields” are currently still in existence, the question arose as to the type of geological conditions under which it would be imperative to use the new standard units. In addition to this, the old shield designs currently in operation still have to be employed to optimum effect for the remainder of their useful life under the given operating conditions.

As part of a research project carried out by Deutsche Steinkohle AG (DSK) a new system was developed for calculating the influence of the measured support resistance on the coal-face roof control process. This roof control process was quantified on the basis of the RFF of the roof beds. The RFF is a yardstick for the tendency of the roof to collapse.
In spite of all the advances in IT development a problem of this type cannot fully be resolved by means of numerical simulation. Such an operation would require universally applicable model concepts to represent the development of the roof-fall process and the ways in which this can be influenced. Even today this information can only be obtained under practical conditions, with underground coal faces acting as real-life test beds.

![FIG. 3 - Physical longwall model](image)

Underground measurements taken on current production faces were therefore used as a basis for the investigations. The caving behaviour of the roof beds under longwall conditions was derived from existing physical models (Figure 3). These model concepts were then examined on the basis of the underground measurements. Only by applying these underground measurements was it then possible to develop regression analyses for determining the roof-fall frequency as an indicator for the actual roof control system.

**LONGWALL DATABASE**

The research project involved taking underground measurements using the “longwall observation method”, which has been a recognised technique for at least 30 years (Figure 4), and then storing this information in a longwall database (Jacobi, 1981). The longwall observation method includes, for example, fixing the position of the face conveyor and supports in relation to the coal face by taking three distance measurements, namely coal face to conveyor, conveyor to tip of skid and canopy tip to coal face. Other measurements taken include the extended shield height, the hydraulic pressure in the two shield legs and the canopy and skid inclination. If roof fall cavities are present, records are taken of their height and width. A data bank of this kind provides a very accurate description of how each shield functions in reaction to the current operating situation. Measurements are not taken from each and every shield along the face line. By surveying every eighth or tenth shield it is possible to obtain a representative picture of the condition of the face supports and roof strata at a certain face position.
**Measured distances, heights & widths**

A: Tip canopy to face (TF)  
B: Conveyor to face  
C: Tip base to conveyor  
D: Shield height  
E: Extension forepole  
F: Height stone cushion  
G: Tip canopy to 1.abutment  
H: Fall height  
I: Fall width  
P: Leg pressure left/right

**FIG. 4 - Underground longwall observation method**

Figure 5 shows the DMT longwall database measurements available for our evaluations. In all a total of 14 production faces were investigated. Some 135 to 315 individual measurements were carried out on each face, making an overall total of 3,137 observations. This body of data represents a representative cross-section of longwalls operating at depths of 800 to 1,200 metres and equipped with a wide variety of different shield support designs.

**No. of underground measurements**

![Graph showing number of underground measurements]

**FIG. 5 - Longwall database**

A number of technical terms, which are essential for understanding the analysis are discussed below (Iresberger, Grawe, Migenda, 1992):

**Roof Fall Frequency FF (%)**

The measured Roof Fall Frequency (RFF\textsubscript{measured}) is the ratio of the number of observed shields with roof falls to the total number of observed shields per face. This value cannot be relayed by remote data transfer to the surface, even when using electrohydraulic shield controls, but can only be measured underground.

The calculated Roof Fall Frequency (RFF\textsubscript{calculated}) describes the probability (in %) that a roof fall will occur at a specific place and is a criterion for the roof-fall sensitivity of the roof beds (Jacobi, 1981, Iresberger, Grawe, Migenda, 1994).

**Theoretical Support Resistance (TSR) (kN/m\textsuperscript{2})**

The TSR is the calculated theoretical support load of a shield in relation to the supported roof canopy surface up to the coal face. When calculating the TSR, different values are used at international level for the support load in kN (leg setting load or yield load), in relation to the roof area in m\textsuperscript{2} (shield in back or in forward position) (Peng, Chiang, 1984).
Measured Support Resistance (MSR) (kN/m²)

The MSR is calculated using measurements obtained underground. This takes account of the measured leg pressure in relation to the actual area of roof surface available. The calculation process includes the roof area, which is determined from the measured face width (specified canopy length plus measured distance between canopy tip and coal face) multiplied by the width of the shield.

The MSR is generally smaller than the more theoretical CSR. However, MSR is of decisive importance for roof control and for roof fall frequency on the face, as it describes the support loads actually transferred to the roof. Our investigations were therefore based exclusively on an evaluation of the measured values MSR.

ROOF FALL GEOMETRY

A good overview of the roof fall geometry is shown in Figure 6, in which the measured height of a roof fall on the face is plotted against the respective distance between roofbar tip and coal face (Jacobi, 1981). It is clear that roof falls only occur when a certain tip to face distance is exceeded – in this case the figure is above 40 cm. The roof-fall height can be all the more considerable, the greater is the distance from canopy tip to coal face. If this critical tip to face interval can be measured, there is a possibility of preventing roof falls. The question now arising concerns the nature of the parameters that affect this critical tip to face distance.

![Diagram of roof fall geometry](image)

**FIG. 6 - Roof fall geometry – relationship between tip to face distance (TF) and height of fall**

CRITICAL TIP-TO-FACE DISTANCE (TF_{crit})

It has been accepted that the tendency of the roof to fail is dependent on the stability of the first series of strata between two solid abutments (Figure 7). These abutments are the coal face and the face supports. As a result, the stability of this beam, and therefore also the critical tip to face distance, is dependent on the thickness and strength of the first roof bed.
The next stage was to compare the strengths and thicknesses of the first roof bed, as determined from strata sections, with the critical tip to face distances established by measurement (Figure 8). The strengths of the investigated roof beds varied from $\beta = 36 \text{ N/mm}^2$ (soft shale) and $\beta = 45 \text{ N/mm}^2$ (shale) to $\beta = 63 \text{ N/mm}^2$ (sandy shale). Regression calculation methods were used to obtain an exponential equation that set the strength and thickness of the first roof bed in relation to the $T_{F_{\text{crit}}}$.

Assumption:
$$T_{F_{\text{crit}}} = f (\text{strength and thickness of 1. layer})$$

FIG. 7 - Critical distance between canopy Tip and coal Face ($T_{F_{\text{crit}}}$)

If the distance to the first stratum change is 70 cm, the critical tip to face distance in the case of sandy shale ($\beta = 63 \text{ N/mm}^2$) is about 1.70 m (Figure 8). On a coal face with such a strong roof it is therefore unlikely that there will be roof fall problems, even at large tip to face distances. However, when the roof strata consists of soft shale ($\beta = 36 \text{ N/mm}^2$) this distance is reduced to a mere 1 m. This means that longwall faces with such a roof composition will be much more prone to roof falls.

This assumption, which has been recognized for many years, has become quantifiable with the acquisition of the regression equation and the general theory now required verification using the measured data. To this effect the following hypothesis was formulated: If the measured tip to face distance is smaller than the calculated value, no roof fall can take place (Figure 9). Of the total of 3,137 data records taken, 2,416 were used for the verification process. In 736 of the cases the measured tip to face distance was smaller than the critical interval. In 728 of these instances (99%) no roof falls occurred. Only in 8 cases (1%) were rock falls recorded. In the 1,680 cases in which the measured tip to face distance was greater than the critical interval a total of 204 roof falls occurred.
(12%). The probability of a roof fall occurring is therefore 12 times greater when the critical tip to face distance, as measured underground, is exceeded. By comparison it can be stated that the probability of a roof fall occurring is very low (1%) when the critical tip to face distance is not exceeded. The above hypothesis is thereby confirmed.

Under a given set of geological conditions the critical distance from canopy tip to coal face is therefore the key quantity for roof control and for selecting the most suitable support system for the existing underground conditions.

**CALCULATION OF ROOF FALL FREQUENCY**

Regrettably, operational conditions on the face mean that it is not always possible to avoid exceeding the critical tip to face distance in every underground situation. This is clarified in Figure 10, which plots roof-fall frequency against the tip to face distance. The measurements shown were taken from 3 longwall production faces on which the critical tip to face distance was approximately 70 cm, 80 cm and 90 cm respectively. Once the critical tip to face distance has been exceeded, however, the increase in roof fall frequency is very variable. The real question is, what influences the increase in roof fall frequency?

**Fall frequency FF > 0 cm (%)**

![FIG. 9 - Hypothesis of TF\textsubscript{crit}](image_url)

![FIG. 10 - Roof-fall frequency gradient](image_url)
In order to answer this it was necessary to investigate a number of additional criteria. The first of these concerned a factor that has long been recognized from rock mechanics engineers over the years, namely the angle between the direction of the fissures and the direction of longwall mining. Figure 11 shows the measured roof fall frequency as a function of the angle between the direction of mining and the direction of the fissures in gon (where $100\text{gon} = 90^\circ$). At an angle of 0 gon longwall-parallel fissures run into the coal face (under the fissures) and beds slipping from the roof are supported by the coal face. At an angle of $+\ or\ -\ 200\ \text{gon}$ the longwall-parallel fissures run into the goaf (on the fissures) and beds slipping from the roof are able to fall into the face cavity in front of the shield canopies. For reasons of simplification the index 1 was used to denote the favourable situation of “under the fissures” and the index 2 was used for the unfavourable situation of “on the fissures”.

The measured roof fall frequencies in the central quadrants are evidently lower than those in the outer quadrants (Figure 11). Here the observed variation in RFF of 15 to 75 % is caused by differences in the other parameters eg MSR and stress. How great the effect of fissure direction is can be illustrated by taking as an example two underground measurements shown in Figure 11. When working 653’s panel (circled) roof fall frequencies of between 15 and 45 % were observed. The adjacent 654’s panel was worked in the opposite direction. Here the roof fall frequency was a maximum of 15 % - the same as the minimum frequency rate for the preceding panel. This is solely due to the change in working direction, since the supports and all other parameters were almost identical in both panels.

A calculation equation was established by a process of regression (Figure 12) incorporating the following parameters:

- calculated vertical rock pressure $p_v$
- measured support resistance MSR
• fissure-direction index DI and
• the value by which the critical tip to face distance is exceeded underground (\( ?TF \))

A calculation equation was established by a process of regression (Figure 12). The definiteness ratio of this equation is 86%, which is a very high figure when it comes to the evaluation of underground measurements. The following parametric studies of the factors involved illustrate how great an influence the latter have on the roof fall frequency.

\[
FF = \frac{c_1 \cdot \frac{p_v \cdot DI}{MSR}}{c_2 \cdot DI \cdot c_3 \cdot (\Delta TF)}
\]

**Determination coefficient = 86 %**

**FIG. 12 - Roof-fall frequency gradient FF – determinant equation variables**

Figure 13 shows the varying increases in roof fall frequency once the critical tip to face distance of 60 cm has been exceeded. While there is a substantial rise in roof fall frequency at a low measured support resistance, this increase is clearly more moderate when the measured support resistance is high. Figure 14 shows the effect of measured support resistance on the roof fall frequency for a favourable (DI = 1) and unfavourable (DI = 2) direction of fissures. While doubling the measured support resistance from 350 to 700 kN/mm\(^2\) has little impact when the fissure direction is favourable, the same operation produces a significant effect when the direction of the fissures is unfavourable.
A case study shows how the information obtained can be put to good effect for the benefit of the user. This depicts the calculated critical tip to face distance for two panels that were investigated using two strata sections obtained from 34 boreholes (Figure 15). Favourable geology means that no strata control problems are encountered in the light shaded area (TF\text{crit} ≥ 1.2 m). However, as the strata series changes unfavourable areas also develop that require a critical tip to face distance of less than 60 cm in order to prevent roof falls. In these areas, which are identified by dark shading in the diagram, the face must either maintain a maximum tip to face distance of 60 cm or – in the event that the critical tip to face distance TF\text{crit} of 60 cm is exceeded – significantly increase its measured support resistance, if it is to avoid roof falls. In certain circumstances this will mean recommending a stronger set of shield supports.

FIG. 14 - Parameter variation – DI to FF and MSR
FIG. 15 - Case study 2 – critical distance between canopy tip and coal face (TF_{crit})

CONCLUSION

Armed with these results DMT is now able to predict the critical distance between the tip of the canopy and the coal face (TF_{crit}), as well as the roof fall frequency, for all shield designs. By applying the new calculation method it is now possible to compare alternative longwall layouts and different shield support types under pre-set geological conditions. Mining engineers on site are therefore in a position to make the necessary roof control preparations required to run the longwall operation to maximum efficiency. The results provide a useful basis for making practical recommendations and for selecting the most effective design of shield support.

REFERENCES

ROADWAY SPAN STABILITY IN THICK SEAM MINING - FIELD MONITORING AND NUMERICAL INVESTIGATION AT MORANBAH NORTH MINE

Baotang Shen¹, Brett Poulsen¹, Michael Kelly¹, Jan Nemcik² & Chris Hanson³

ABSTRACT: Comprehensive field monitoring and numerical analyses of roadway span behavior have been conducted at Moranbah North Mine as part of an ACARP Project for Rapid Roadway Development. The response of the coal roof to roadway development in a 5.5m thick seam was studied. The monitoring program was highlighted by successfully measuring the roof stress change prior to, during and after the roadway excavation. Roof displacement was also monitored after the installation of roof support.

Measured in situ stresses in the coal seam were much lower than estimated from the overburden depth. The monitored vertical stress in the coal roof exhibited a rapid drop immediately after the roadway face passed the monitored locations. It however, recovered substantially (up to 50%) in the next 10 hours. Two roadway sections 60m apart were monitored. The monitored roof displacements and stresses in the two locations showed a significant difference. The sites were separated by two normal faults.

The monitored roadways were simulated using fully coupled mechanical-flow models. The numerical results suggested that the drainage of gas/water around the roadways was likely to be the main cause of the observed stress recovery in the coal roof. It was also found that the presence of high gas/water pressure in the coal seam would result in lower stress readings with the stress measurement technique. The measured stress is close to but not necessarily equal to the effective stress.

INTRODUCTION

A research project sponsored by JCOAL/CSIRO and ACARP is in progress to develop an automated roadway development system. The system consists of an Autonomous Conveying/Bolting Machine (ACBM) and a continuous miner. The system will enable a continuous and rapid roadway advance. Successful application of the system requires stable roadways before full support can be installed. The potential unsupported (or lightly supported) span inbye of the system is about 12m. It is important that the stability of unsupported spans in various geotechnical conditions are fully understood to assess the applicability of the Rapid Roadway Development System in different mines. This paper will focus on the study of coal roof behaviour for the application of the ACBM system in thick seam mining.

It is known that the behaviour and failure mechanisms of unsupported spans are complex, affected by many factors such as rock strength, roof geology, stress and roadway geometry. Strata Engineering Pty Ltd (Frith, 1998) considered that span failure is dominated by beam bending or shearing; Seedsman (2001) suggested that span instability often starts from shear failure of the roof bedding planes. Strata Control Technology Pty Ltd (SCT) (Gale, 1991) considers the roof to be a yielding material and continuum numerical modelling has been used to assess the span stability. Shen and Duncan Fama (1996, 1999) studied the failure mechanisms of unsupported spans of highwall mining entries and proposed analytical methods for their stability assessment.

Many Australian longwall mines operate in thick coal seams and leave coal in the roof on development. The behaviour of a coal roof differs from that of bedded rocks due to the coal cleating, low coal strength, possible low stress environment, and high gas content. The understanding of coal roof behaviour is limited, and requires further investigation.

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² Strata Control Technology Pty Ltd
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With the support of Moranbah North Mine and help from SCT, CSIRO has conducted a comprehensive field and numerical study of coal roof behaviour at Moranbah North. Two sections of the Longwall 104 gate roads at Moranbah North were monitored and investigated. The study consisted of the following components:

- Geological and geotechnical investigation of the roof conditions in the monitored area.
- Determination of the strength of roof materials.
- Monitoring of the stresses and displacement in the roadway roof strata.
- 3D mechanical numerical modelling.
- 2D coupled mechanical and fluid flow modelling, including impact of increased mining depth.
- Roadway stability assessment for the potential application of the ACBM system.

**GEOLOGICAL AND GEOTECHNICAL CONDITIONS**

Moranbah North Mine is located at the Bowen Basin Coal Field, Central Queensland. The primary mining seam is the Goonyella Middle (GM) seam which varies in total thickness across the mining resource from 5.2m to 6.4m, with a total thickness in the study area of approximately 5.5m. The seam comprises moderately weak coal which is dull at the top and grades to dull and bright banded, and is bright banded at the base. The GM seam has an estimated gas content of 5 m$^3$/t at the depth of current roadway development.

The gateroads at Moranbah North Mine are nominally cut to 5.2m wide and 3.0 – 3.2m high, excavated in the lower part of the seam. Current development cuts to the floor of the coal seam and is designed to leave at least 1m of coal in the immediate roof. The stability of the roof is believed to be strongly dependant on the thickness of coal left in the immediate roof. The place-change roadway development method is used. The typical cut-out distance ranges from 6m to 15m depending upon the local roof conditions and the overburden depth.

A decision was made to monitor roadways in 104 panel. As a preliminary site investigation, five cored boreholes were drilled into the roof and coal ribs at 47C/T and 49C/T to investigate the geological and geotechnical conditions at the monitored sites. The cores were geologically and geotechnically logged. Samples were selected for laboratory testing to determine the strength of coal and rock material.

The geology from the drill holes is generally consistent. The typical roof geology in the monitored area is shown in Figure 1. The immediate roof is 1.7m coal. It is of C3 type (40-60% bright) immediately above the roof and becomes dull coal with bright bands at the top of the seam. Overlying the coal seam is a weak to very weak mudstone unit. The mudstone unit has a thickness of about 1.8m. Above this unit is the Goonyllia Rider Seam (GMR) which has an average thickness of 0.2m. Above the GMR unit is a sequence of mudstone/siltstone/sandstone units.

Also shown in Figure 1 is the Coal Mine Roof Rating (CMRR) for different roof units. The roof coal and the mudstone units have a CMRR value $\leq$ 35 (Figure 1). According to Mark (1999), unsupported roadway spans with a CMRR less than 46 are considered to be unstable in the long term.

Fourteen coal and rock samples selected from the roof cores were tested in uniaxial compression to determine the strength and deformability (Cunnington and Boland, 2002). The tested Uniaxial Compressive Strength (UCS) and Young’s modulus (E) are also plotted in Figure 1.

The average UCS and Young’s modulus of the roof coal and rocks are summarised in Table 1.

**Table 1. Strength and deformability of roof units, based on laboratory tests results**

<table>
<thead>
<tr>
<th>Roof unit</th>
<th>UCS (MPa)</th>
<th>Young’s modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate roof coal:</td>
<td>8.0</td>
<td>2.2</td>
</tr>
<tr>
<td>Mudstone:</td>
<td>Weak, no samples were recovered</td>
<td></td>
</tr>
<tr>
<td>GM Rider Seam:</td>
<td>10.5</td>
<td>3</td>
</tr>
<tr>
<td>Mudstone/Siltstone</td>
<td>13.0</td>
<td>2.8</td>
</tr>
<tr>
<td>Sandstone (lower part):</td>
<td>19.5</td>
<td>3.7</td>
</tr>
<tr>
<td>Sandstone (upper part):</td>
<td>59.5</td>
<td>14.6</td>
</tr>
</tbody>
</table>
FIELD MONITORING

Two sections of Roadway A at LW 104 (Figure 2) were monitored in the vicinity of the 51 and 52 C/Ts where overburden depth is 220m. The monitoring program included three main components:

- Monitoring of stress change in the roof during roadway development;
- Monitoring of roof displacements after the excavation of the roadway; and
- Measurements of in situ stresses in the vicinity of the monitored area.

The stress change monitoring was designed to record the stress change in the roadway roof prior to, during and after the roadway excavation. This method overcame the shortcomings of the conventional displacement monitoring techniques which can only monitor the roof response after the excavation of the roadway (and usually after bolting), missing the period when the crucial roadway reaction is occurring.

Four ANZI stress cells were installed at Site 1 and two at Site 2, see Figure 3. The stress cells were located in different roof units and they were installed in boreholes drilled from the cut-throughs prior to the excavation of the monitored roadway sections. Figure 4 shows one such arrangement. After the installation of the stress cells, the roadways were cut in the sequence marked in Figure 4 i.e. 1→2→3→4. The changes of the stresses in the roof were monitored with frequent data reading (every 30 minutes) when the monitored roadway sections were cut.

Four sonic extensometers were installed in the centre of the roadway in the monitored section at each site. The extensometers were installed after the completion of the roadway cutting and roof bolting.
The *in situ* stresses in the coal seam and roof rock were measured in the vicinity of the monitoring sites using the overcoring technique. The *in situ* stress measurements were conducted in the pillar between 49 and 50 cut-throughs (SCT, 2002).

**FIG. 2** - Mining layout and the monitored sections of the roadway heading A.

**FIG. 3** - Location of stress cells in roof at the two monitoring sites
In situ stresses

The *in situ* stress measurement results are given in Table 2. The *in situ* stresses in the coal seam are much lower than that estimated from overburden depth. The measured vertical stress is only 1.3 MPa, compared with the calculated vertical stress of 5.5 MPa from overburden depth of 220 m. It is believed that the measured results have been affected by gas/water drainage around the measurement borehole. The measured stress is likely to be close (but not necessarily equal) to the effective stress (Shen, Poulsen and Kahraman, 2002).

Table 2  Summary results of the *in situ* stress measurements

<table>
<thead>
<tr>
<th>Strata</th>
<th>Principal stresses</th>
<th>Magnitude (MPa)</th>
<th>Dip (degrees)</th>
<th>Strike from North (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>$\sigma_1$</td>
<td>1.83</td>
<td>60</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td>$\sigma_2$</td>
<td>0.46</td>
<td>10</td>
<td>245</td>
</tr>
<tr>
<td></td>
<td>$\sigma_3$</td>
<td>-0.27</td>
<td>28</td>
<td>152</td>
</tr>
<tr>
<td></td>
<td>$\sigma_v$</td>
<td>1.3</td>
<td>90</td>
<td>0</td>
</tr>
<tr>
<td>Roof sandstone</td>
<td>$\sigma_1$</td>
<td>13.97</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>$\sigma_2$</td>
<td>7.94</td>
<td>31</td>
<td>271</td>
</tr>
<tr>
<td></td>
<td>$\sigma_3$</td>
<td>5.95</td>
<td>59</td>
<td>105</td>
</tr>
<tr>
<td></td>
<td>$\sigma_v$</td>
<td>6.5</td>
<td>90</td>
<td>0</td>
</tr>
</tbody>
</table>

Stress changes in roadway roof

The measured stress changes in the roof coal, mudstone and sandstone during roadway advance are shown on a vertical section in Figure 5. The vertical stress change in the roof coal with time is shown in Figure 6.

Stresses in the roof rotated when the roadway face approached and passed the monitored locations (FIG. 5). At Site 2, cell 15 in the Rider Seam indicated a change in level and orientation implying shearing along the Rider Seam. Vertical stress in the roof dropped sharply immediately after the roadway face passed the monitored locations (Figure 6).
After the initial stress drop, the vertical stress recovered partially in the next 2-3 hours in the GM Seam (Figure 6) and the mudstone unit. The recovery continued in the next 12 hours at a reduced rate. No vertical stress recovery was observed in the sandstone roof.

The magnitude of the vertical stress change in the GM Seam at Site 1 was generally consistent with that of in situ vertical stress obtained from the overcoring stress measurements. The maximum vertical stress drop is about 1.2-1.3MPa, while the measured in situ vertical stress in coal was 1.3MPa. This implies a nearly total release of vertical stress after roadway excavation.

At Site 2, the maximum vertical stress drop in coal roof is about 2MPa, much higher than that at Site 1 and the in situ vertical stress measured in the coal pillar between 49 C/T and 50 C/T. The results suggest that Site 2 had a different stress regime from Site 1, and the in situ stresses were higher. The difference might be influenced by unknown local geological variations and the existence of geological structures. Two normal faults exist between Site 1 and Site 2. The faults are nearly perpendicular to the roadway axis, both dip in an angle of 70 degrees inbye. One of the faults has no obvious displacement, and the other has a displacement of 0.2m. The fault surfaces are clean and tight. Minor displacement normal faulting of this nature is not uncommon at Moranbah North.

Roof displacement

Four sonic extensometers were installed in the monitored roadway roof immediately after roof bolting at each site. Displacement data were read as the roadway advanced. The maximum displacement was obtained at the monitored location closest to the cut-throughs (Extensometer 1 at Site 1, see Figure 4, and Extensometer 5 at Site 2). The two extensometers were installed close to the face of previous cut and therefore measured the full roof response to the roadway advance. Other extensometers were installed within the length of the full bolted roadways where much of roof displacement might have already occurred. They hence measured much less roof displacement. The monitored roof displacement from Extensometers 1 and 5 are shown in Figure 7.

A maximum of 18mm roof movement was recorded at Site 1 (Figure 7a). The recorded displacement is relative to the upper end of the extensometers (7m into roof). The deformation zone extended about 4m into the roof. It is apparent that the bolted roof remained intact as the deformation in the roof coal layer was almost uniform.

The Extensometer 5 at Site 2 recorded a maximum of 50mm roof displacement (Figure 7b), substantially higher than that at Site 1. The deformation zone extended at least 6m into the roof, also higher than that at Site 1.

The larger roof movement at Site 2 is consistent with the higher stress change at this location. The roof displacement data support the suggestion that a different stress regime existed at Site 2 which is separated from Site 1 by two normal faults.
FIG. 5 - Monitored stress change in roof coal and rock units versus roadway face position in a vertical cross section parallel to the roadway axis. Outward arrow indicates reduction of compressive stresses.
FIG. 6 - Change of vertical stress in the roof coal at the two monitored sites.

**Vertical stress change in roof coal at two sites**

![Graph showing vertical stress change in roof coal at two sites.]

**SITE 1: Maingate 104 A Heading 51CT+4m Roof Extensometer 1**

- 29.10.01 21:30
- 29.10.01 22:35
- 30.10.01 2:30
- 30.10.01 11:55
- 31.10.01 8:40
- 30.10.01 16:50
- 07.11.01 8:40
- 27.11.01 13:30

**SITE 2: Maingate 104 A Heading 52CT+9m Roof Extensometer 5**

- 06.11.01 11:00
- 07.11.01 8:30
- 09.11.01 8:30
- 27.11.01 13:30

**FIG. 7 - Roof displacement at the two monitored sites.**

![Diagram showing roof displacement at two monitored sites.]

**NUMERICAL MODELLING**

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Numerical modelling was conducted using two and three dimensional models. The aim of the numerical modelling was to reproduce and interpret the measurements and hence to better understand the roof behaviour at the monitored sites.

The three dimensional numerical simulation of the monitored roadway was conducted used FLAC3D (Itasca, 1997). A 3D numerical model was set up based on the simplified geological log shown in FIG. 1. The true mining sequence, including roof and rib bolting, was simulated. The modelled stress change and displacements in the roof were investigated and compared with the measurements. The model geometry is shown in figure 8.

The two dimensional numerical modelling used FLAC with full mechanical-fluid flow coupling (Itasca, 1999). Roof and rib bolting were installed 5 hours after the cut of roadway section. The initial gas and water pressure in the coal seam was assumed to be 3.0MPa. Fluid flow in the coal seam during roadway excavation was modelled. For simplicity, the gas and water were treated as the same fluid medium and hence only single phase fluid was modelled.

The predicted stress changes in the coal roof from the 2D and 3D modelling are plotted in Figure 8.
(a) 3D mechanical modelling results

FIG. 9 - Modelled vertical stress change (un-marked lines) in roadway roof at Site 1 in comparison with the measurements (marked lines). The modelled results are given at different distances into the roof. Excavation steps correspond to the roadway sections marked in Figure 4. Steps 2a-2d are excavations of the 10m section 2 with 2.5m increment, 2e is after roof bolting. Step 5 is further advance of the roadway.

Using the 3D model without considering gas/water pressure, the predicted change of vertical stress in the coal roof during roadway excavation was about 5.5MPa, much higher than measured (see Figure 9a). Unlike the measurements, the model predictions did not show any recovery of the vertical stress in the coal roof after the initial drop.

The 2D model simulated the gas/water pressure and fluid flow in the coal seam during the excavation process. Figure 10 shows the modelled pore pressure distribution 5 hours after roadway excavation. The predicted vertical stress change in the coal roof was in general agreement with the measurements (Figure 9b). The predicted maximum magnitude of the vertical stress change was 2.5-3.0MPa, closer to the measurements of 1.3MPa at Site 1. More importantly, the modelling results showed a clear stress recovery after the initial stress drop, consistent with the measurements.

(b) 2D coupled modelling results

FIG. 10 - Pore pressure distribution around roadway, 5 hours after excavation
The modelling results demonstrated that, in thick seam mining where gas and water pressure exert an influence on the overall properties of the coal seam, the pore pressure and fluid flow could play an important role in the behaviour of the roadway. The effects of pore pressure and fluid flow need to be considered in the stability assessment of the roadway spans.

Additional modelling was conducted using the 2D coupled mechanical-fluid flow model at different depths. The modelled roof displacement at two depths (220m and 300m) are plotted and compared in FIG. 11. The case of increased depth will also be relevant to the Site 2 condition where higher in situ stress existed, equivalent to a deeper location.

At the current depth of 220m where the measurements were conducted, the predicted maximum roof displacement is 19 mm. The predicted roof displacement is in a close agreement with the measurements from roof Extensometers 1 (FIG. 7a). The model predictions also show a clear time variation due to the fluid flow after excavation.

At the depth of 300m, the predicted maximum roof displacement is 32mm, significantly higher than that at the depth of 220m.

CONCLUSIONS

The main results of the study are summarized below for the site specified conditions at Moranbah North Mine.

- The monitored roadways at a depth of 220m were stable. However, the roadways at the two monitoring sites 60m apart showed significantly different roof behaviour. The roadway at the second site showed significantly larger roof displacement (50mm) than that at the first site (18mm). The monitored stress change in the roof at the second site was also higher than that at the first site.
- Major geological structures may have affected the roadway roof conditions in the monitored area. The two monitored sites were separated by two normal faults, perpendicular to the roadway axis. The
faults have up to 0.2m displacement and their surfaces are tight. This type of fault is not uncommon in the mine. The monitoring results suggest that the existence of faults has changed the stress regime and altered the mining conditions.

- The measured in situ stresses in the coal seam are much lower than those estimated from the overburden depth (the measured vertical stress of 1.3MPa compares with 5.5MPa estimated from overburden depth). The gas/water pressure in the coal seam would have affected the measurement results. The measured stress is close (but not equal) to the effective stress.
- The vertical stress in the immediate coal roof is found to drop immediately after roadway excavation. The magnitude of the stress drop suggests that the vertical stress in the immediate roof may have become tensile immediately after the roadway excavation. It is likely that roof delamination would have occurred in the immediate roof before the installation of roof bolts.
- After its initial drop, the vertical stresses in the coal roof are found to recover substantially (up to 50%) within 10-12 hours after excavation. Numerical modelling results suggest that the stress recovery was caused by the drainage of gas/water and the redistribution of the stresses around the roadway.
- It is geotechnically feasible to apply the ACBM system at the current depth in Moranbah North Mine. It is emphasized however, that geotechnical conditions (including in situ stresses) may vary significantly, particularly where faults exist. The localized deterioration needs to be considered in mining planning and design.

It is emphasized that, when mining in thick seams particularly with high pore pressure, fluid flow has significant impact on the stability of the roadways. It is recommended that the effects of pore pressure and fluid flow be considered during roadway support design and strata control. The study results also demonstrated the need for a better understanding of the stress and stress measurement results in the coal seam. A clear, quantitative relation between the measured stress and the effective stress/total stress in the coal seam should be developed for the conventional stress monitoring techniques.

ACKNOWLEDGEMENTS

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REFERENCES

ABSTRACT: Within the mining industry, there is recorded data which describes damage to people. The damage can be classified in a hierarchy ranging from incidents which produce multiple fatalities, single fatalities, non-fatal permanent disabilities, temporary damage and inconvenience. There is also a body of unreported collective knowledge in the workforce associated with the tasks which do not produce damage, which are not perceived as high risk; are seldom, if ever, reported and are consistent with that which produce non-fatal permanent damage. The key to successful high risk management is the collection of this unreported knowledge.

The vast majority of personal damage (measured in dollars, suffering, and impairment) is associated with non-fatal permanent disability. It is multiple fatalities and single fatalities which bring about the greatest level of change through the attention which is drawn to such events but these are not the categories which produce the majority of damage. Organisations must predict the potential for permanent non-fatal damage within their operation.

The mining industry’s pattern of non-fatal permanent disability has been accurately described. This generalised pattern provides the basis of implementing systematic high risk identification using appropriate focusing questions and focused groups comprised of underground miners. The process is known as Focused Recall. It is a systematic collection of the experience and knowledge of the workforce against the pattern of non-fatal permanent disability. It couples appropriate experience with external expertise. The process has been applied to Oaky Creek Coal and Oaky Creek North and, in particular, their longwall operations, the development crews and support groups. The pattern of collected data parallels the known industry pattern of non-fatal permanent disabilities. The process harvests the collective experience and knowledge which has seldom, if ever, been reported into the organisation’s data base. The information correlates strongly with the phenomena of non-fatal permanent disability.

The results of the use of this powerful productive process at Oaky Creek Coal and Oaky Creek North are presented.

INTRODUCTION

Work related non-fatal permanent damage is by far the greatest cost to the community, the family and the individual. Regardless of whether the cost is measured in terms of dollars, pain, impairment (a medical judgement of the percentage loss of function) or emotional hardship, non-fatal permanent disability is the most significant category of personal damage. The future prediction and management of this level of personal damage should be of the highest priority. Very seldom is there a lack of physical and financial resources to achieve change once the issues have been identified.

The critical issues appear to be:

1. A recognition of the size and nature of the personal damage problem;
2. Collating the future potential exposure for the particular mining operation into a manageable document.

An appropriate focus (the elimination of permanent personal damage) with an appropriate predictive strategy is the key to progress.
THE SIZE AND NATURE OF THE PROBLEM

Personal damage can be caused by those aspects of work which produce:

- Multiple fatalities;
- Single fatalities;
- Non-fatal permanent damage;
- Temporary damage;
- Minor damage;
- Reported near-misses.

The majority of experience at sites is with the latter three categories. It is necessary to establish what is the relative size (numbers of people) and cost of the different categories. Probably the most definitive work is that documented by the Commonwealth Department of Training’s Industry Commission Report into Workplace Health and Safety (1995). They categorised damage as:

- Less than five days off work;
- Five days and more off work and return on to work full duty;
- Five days and more off work and return to work on reduced duties for a temporary period;
- Invalided out and return to work after a long period of absence on a permanently reduced income (permanent damage)
- Permanently incapacitated and does not return to work
- Fatality

Table 1 shows the number of cases and cost of damage for Australia 1992-1993.

Table 1 Number of Cases & Cost of Damage

| NUMBER OF CASES & COST OF DAMAGE (Australia 1992 – 93) |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| <5 days | >5 days, full duties | >5 days, reduced duties | Permanently incapacitated | Fatal |
| No. of occurrences | 144,953 | 123,395 | 78,333 | 30,728 | 19,290 | 693 |
| % of occurrences | 30.33 | 31.12 | 19.75 | 6.74 | 4.86 | 0.17 |
| Cost of occurrences (Billions) Total 20 | 0.136 | 1.063 | 2.415 | 4.555 | 11.684 | 0.299 |
| % of cost | 0.67 | 5.28 | 11.99 | 22.82 | 57.93 | 1.48 |

Basically the table can be summarised as follows:

(a) From a total of 396,492 occurrences:
- 50,711 occurrences are permanent, and
- 345,800 occurrences are temporary damage.

(b) Of the 50,711 permanent damage incidents:
- 693 were fatal
- 19,290 were non-fatal – no return to work and
- 30,728 were non-fatal – reduced income work.
The total cost derived from direct and indirect costs is $20 billion and is allocated to:

- permanent – fatal $0.3 billion
- permanent – non-fatal $16.4 billion, and
- temporary - $3.6 billion.

This data is based upon 1992-1993 figures. There is no equivalent dataset produced by any authority in Australia since that time which gives such a clear distinction between the different categories of personal damage. Essentially the Pareto Principle applies: 80% of the damage is associated with 12% of the incidents.

To determine whether there has been any significant change in the size and nature of the problem, reference is made to WorkCover New South Wales Statistical Bulletin 1999-2000 (2001). This organization usefully categorises damage as fatal, permanent disability, temporary disability - greater than six months and temporary disability - less than six months. It is useful to understand that a person who has been off work for more than six months has a one in four chance of returning to work, and a person who has been off work for more than twelve months would have a one in two chance of returning to work. These ratios are now applied to published incident data to gain insight into the current size of the permanent damage problem.

New South Wales datasets define non-workplace injuries as being caused by accidents “occurring away from the workplace but where the worker is considered to be on duty eg road traffic accidents”. Workplace injury refers to an accident “which occurs at the workplace either during work or during a work break”. The results for the year 1999-2000 can be summarised as follows in Table 2:

<table>
<thead>
<tr>
<th></th>
<th>Permanent Disability</th>
<th>Six Months &amp; Over</th>
</tr>
</thead>
<tbody>
<tr>
<td>Workplace injuries/permanent disability</td>
<td>8 818</td>
<td>3 951</td>
</tr>
<tr>
<td>Non-workplace injuries</td>
<td>995</td>
<td>550</td>
</tr>
<tr>
<td>Total Numbers</td>
<td>9 813</td>
<td>3 951</td>
</tr>
</tbody>
</table>

With respect to Table 2, assuming a one in four people for return to work for the “six months and over cases”, then 10 800 would be categorised as permanent damage cases within New South Wales in one year. If we assume New South Wales represents one quarter to one fifth of the Australian injury problem (a reasonable proposition) then the number of Australian work related permanent damage cases (excluding disease) would still be at least in the order of 50 000 cases. The previous discussion did not include the approximate 5 500 people categorised as permanently damaged from occupational disease. The majority of these cases are associated with noise (80%). The key learning is that there can still be no successful argument to say the size of the permanent damage problem has decreased. There is, in part, an argument to the contrary i.e. the size of the permanent damage problem has increased.

Examination of the National Occupational Health and Safety Commission’s database (1997) for the years 1996-2000 reveals that there were 1 170 cases in the Australian coal mining industry where people experienced more than sixty days off work. Again it can only be a judgement, but assume that one third of those cases are non-fatal permanent damage. This would indicate that the coal mining industry for Australia has experienced approximately 100 people permanently damaged per annum as a consequence of work. Part of the problem, and part of the tragedy, is that non-fatal permanent disability cannot be accurately described.

In the coal mining industry of New South Wales in the period 1998-2000, there were recorded 28 cases of permanent disability and 117 cases of people experiencing six months or more off work. Again applying the 1:4 ratio to the people with more than six months off work there are 57 cases of permanent disability recorded in New
South Wales compared with the recording of 11 cases in Queensland for a similar two year period. It would be nonsense to suggest that Queensland is five times “safer”. The open cut and underground mining populations are similar with Queensland having 8 500 employees and New South Wales 9 606 (Minerals Council of Australia: 2002). New South Wales has a higher percentage of underground employees.

Why does this situation of such low recorded numbers occur? It is that people filter out of the system because non-fatal permanent damage is, in the main, not a traumatic injury; that is, does not involve amputation or disfigurement? If one were to examine 1 000 people who were classified as non-fatal permanent damage the Pareto Principle would apply, i.e. more than 80% of those people would have soft tissue damage to their body structures such as ligament, vertebral disc and tendons. Those people would appear “normal” until the body was asked to do work.

THE LIKELIHOOD OF PERMANENT DAMAGE

Risk can be defined as the product of a particular consequence against the likelihood of that particular consequence. One way of expressing likelihood is in terms of the number of employee years required to produce one case of the particular consequence, essentially an “incident rate”. Based on an Australian working population of approximately 8 million, and 50 000 cases of non-fatal permanent damage, the Industry Commission Report would suggest that the likelihood of non-fatal permanent damage is one per 160 employee years.

For the coal industry, the Minerals Council Safety and Health Performance Report (2002) indicates that the total number of people employed in the Australian coal industry is 20 230. If one accepts the previous statement that there are 100 cases of permanent damage generated per annum then the likelihood of non-fatal permanent damage is one per 200 person years worked the likelihood of fatality within the coal industry 1998-2001 was approximately one per 4600 employee years worked.

How does the likelihood of non-fatal permanent damage for the coal industry compare with New South Wales industry generally? Table 3 shows the likelihood of non-fatal permanent damage, assuming that one in four of those people who are off work for more than six months become classified as permanently damaged.

Table 3 Likelihood of Non-Fatal Permanent Disability – NSW – All Industries

<table>
<thead>
<tr>
<th>Year</th>
<th>Likelihood</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991-1992</td>
<td>1:578</td>
</tr>
<tr>
<td>1992-1993</td>
<td>1:510</td>
</tr>
<tr>
<td>1993-1994</td>
<td>1:361</td>
</tr>
<tr>
<td>1994-1995</td>
<td>1:273</td>
</tr>
<tr>
<td>1995-1996</td>
<td>1:288</td>
</tr>
<tr>
<td>1996-1997</td>
<td>1:262</td>
</tr>
<tr>
<td>1997-1998</td>
<td>1:262</td>
</tr>
<tr>
<td>1998-1999</td>
<td>1:262</td>
</tr>
<tr>
<td>1999-2000</td>
<td>1:255</td>
</tr>
</tbody>
</table>

The interesting observation from this table is that the likelihood is increasing over time with respect to non-fatal permanent damage. The coal industry would not appear to perform any better than industry as a whole. It is suggested that the previous likelihood of non-fatal permanent disability for the coal industry is significantly under-stated. It is possible for a coal mining operation to gather experienced and long-standing employees and make a list of the number of employees (who either still work with the organization or are separated from the organization) carry work-related permanent impairment and complete the calculation set out in Figure 1:
The previous discussion is intended to create sensitivity to the need to identify and predict the future potential non-fatal permanent damage within an organization. Industry has an appropriate but excessive emphasis on catastrophic failure to the detriment of people who are permanently damaged.

**WHAT DO WE KNOW ABOUT THE PATTERN OF NON-FATAL PERMANENT DAMAGE?**

Damage to people can be considered to be a consequence of an energy exchange. Energy is simply the capacity to do the work. Damage to people occurs when the energy exposures exceed the tolerable limits of the person. Energy can be loosely, but usefully, classified as shown in Table 4:

<table>
<thead>
<tr>
<th>Table 4 Damaging Energy Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Human Energy</td>
</tr>
<tr>
<td>Gravitational Energy</td>
</tr>
<tr>
<td>Vehicular Energy</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Machine Energy</td>
</tr>
<tr>
<td>Object Energy</td>
</tr>
<tr>
<td>Electrical Energy</td>
</tr>
<tr>
<td>Thermal Energy</td>
</tr>
<tr>
<td>Chemical Energy</td>
</tr>
<tr>
<td>Noise Energy</td>
</tr>
<tr>
<td>Other Energy sources</td>
</tr>
</tbody>
</table>

An energy exchange can be considered to have a time/intensity relationship (dose) and has been grouped by McDonald & Associates into one of three classifications as listed below in Figure 2.

---

**FOR YOUR SITE**

Average number of Employees over last 5 years = 
Number of permanent disability injuries over last 5 years = 
Likelihood of permanent disability in any one year = Number of Cases 
Number of employee years

**FIG. 1 - Likelihood of Permanent Damage**
FIG. 2 - TimeVs Energy Graphs

Type A Damage
Single traumatic energy exchange.
Examples:
- Electric Shock;
- Hit by fast moving object;
- Burnt by flames;
- Jolt/jar

Type B Damage
A series of discreet energy exchanges, each not affecting the function or generating pain, but each exchange reducing the damage limit. The cumulative effect is damage.
Examples:
- Lifting, pushing or pulling tasks leading to back damage.

Type C Damage
Continuous exposure to small energy exchanges which produce cumulative damage.
Examples:
Continuous exposure to:
- Repetitive movements leading to repetitive strain injuries (occupational overuse syndrome);
- Noise;
- Prolonged postural displacement;
- Chemicals, or
- Ride vibration leading to back damage.

Analysis of datasets within Australia reveals very consistent patterns with respect to non-fatal permanent damage. Human, gravitational and vehicular energy are those few energy sources that contribute 80% of permanent disability.
Table 5 Classification of Accidents in the Coal Mining Industry – New South Wales & Queensland

<table>
<thead>
<tr>
<th>DAMAGING ENERGY</th>
<th>NUMBER OF PEOPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Human Energy</td>
<td>491</td>
</tr>
<tr>
<td>Gravitational Energy</td>
<td>398</td>
</tr>
<tr>
<td>Machine Energy</td>
<td>254</td>
</tr>
<tr>
<td>Object Energy</td>
<td>36</td>
</tr>
<tr>
<td>Thermal Energy</td>
<td>16</td>
</tr>
<tr>
<td>Chemical Energy</td>
<td>12</td>
</tr>
<tr>
<td>Susceptible Part</td>
<td>7</td>
</tr>
<tr>
<td>Anxiety/Stress Disorder</td>
<td>7</td>
</tr>
<tr>
<td>Oxygen Deprivation</td>
<td>1</td>
</tr>
<tr>
<td>Heart Attack</td>
<td>1</td>
</tr>
<tr>
<td>Biological Energy</td>
<td>1</td>
</tr>
<tr>
<td>Specialised Shape</td>
<td>2</td>
</tr>
<tr>
<td>Insufficient Information</td>
<td>5</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>1231</strong></td>
</tr>
</tbody>
</table>

Table 5 is a summary classification of 1,231 cases of permanent non-fatal damage for the New South Wales and Queensland coal mining industries between 1990 and 1995 (892 underground cases; 339 open cut cases). The sponsors for this work were the New South Wales Minerals Council and the Queensland Mining Council. It remains one of the most definitive works with respect to non-fatal permanent disability for the coal mining industry. Table 6 shows the very high involvement of “human”, “gravitational” and “vehicular” energy for the underground coal mines of Queensland and New South Wales. For the period 1990-1995, there were 892 cases of permanent damage. The underground classifications are summarised in Tables 7 to 9:

Table 6 Underground Classification – Permanent Cases 1990-1995

![Diagram of underground classification]

- **Total**: 892 people
  - **Human**: 375 people
    - Gravitational: 309 people
    - Vehicle: 144 people
    - Object: 27 people
Table 7  Gravitational Energy

Gravitational
4 Categories - 73% of Cases
• Falls of rock and stone - 86 people
• Falls from height (work on equipment) - 28 people
• Descending equipment - 35 people
• Slip and trip - 77 people

Table 8  Human Energy

Human
5 Categories - 70% of Cases
• Lifting - 113 people
• Push/Pull - 39 people
• Walking (near fall) - 58 people
• Impact - 36 people
• Descending (near fall) - 14 people

Table 9  Vehicular Energy

Vehicular
1 Category - 60% of Cases
• Jolt/Jarr - 58 people

In these tables, permanent damage is categorised as 90 days or more of work lost. The power of this study is that it brings emphasis; it allows for appropriate focus because of the better describers present in the pattern analysis. The previous tables can be compared to statistics on workplace injuries in the Coal Mining Industry New South Wales for 1998-2000 as shown in Table 10.
### Table 10  Mechanism of Injury for Permanent Disability & “Six Months Plus” cases in Coal Mining Industry – New South Wales 1998-2000

<table>
<thead>
<tr>
<th>Mechanism of Injury</th>
<th>Disease</th>
<th>WorkPlace Industry anzsic 110: Coal Mining</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Permanent Disability</td>
</tr>
<tr>
<td>Falls from a height</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Falls on the same level</td>
<td>29</td>
<td>-</td>
</tr>
<tr>
<td>Hitting stationary objects</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Hitting moving objects</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rubbing and chafing</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Being hit by falling objects</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Being trapped by moving machinery</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Being trapped between stationary and moving objects</td>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>Exposure to mechanical vibration</td>
<td>17</td>
<td>9</td>
</tr>
<tr>
<td>Being hit by moving object</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>Muscular stress while lifting, carrying or putting down objects</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>Muscular stress while handling objects other than lifting, carrying or putting down</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Muscular stress with no objects being handled</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Contact with hot objects</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Slide or cave in</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>Vehicle accident</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Unspecified mechanism of injury</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>TOTAL</td>
<td>119</td>
<td>30</td>
</tr>
</tbody>
</table>

There is no strong focus on jolting and jarring; the damaging phenomena is possibly camouflaged in “exposure to mechanical vibration”. There is no strong focus on issues associated with descending equipment. There is an industry sensitivity to falls of rock and stone which could be either represented in either the “slide or cave in” or “being hit by falling object” classifications of the Table 10.

If non-fatal permanent disability and the understanding of the mechanism of damage is the “signal” to be received and everything else is “noise”, then the type of information presented in the previous Table 10 decreases the “signal” to “noise” ratio and does not allow for an appropriate level of discernment.

The problem is even compounded when sites review their own databases which are sure to incorporate less than five days of lost time. In Queensland in 1996 there was a change in our legislation and the employer paid for the first five days. The number of claims reduced from approximately 46 000 to 26 000 (not including travel claims) however, the review of datasets that contain lesser injuries e.g. less than five days, allow for less discernment. Table 11 is a summary of the Queensland injury database, excluding disease, less than five days for the year 1995-1996. This is the last year in which such data is available. “Eyes” and “heads” are 42% of all injuries yet seldom appear in the non-fatal permanent disability studies.
Examination of the data for lower level injuries indicate that the “signal” to “noise” ratio is such that an appropriate focus is lost. Site incident data is likely to contain those incidents which distract from a recognition of the pattern of non-fatal permanent damage and its implications for management risk.

**HOW TO OBTAIN THE NECESSARY EMPHASIS**

The incident triangle has been a descriptive statistic used in safety literature and safety training for many years. It is illustrated in Figure 3.

<table>
<thead>
<tr>
<th>Body Location</th>
<th>Percentage of Claims Less Than Five Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eyes</td>
<td>13%</td>
</tr>
<tr>
<td>Head</td>
<td>3%</td>
</tr>
<tr>
<td>Neck</td>
<td>4%</td>
</tr>
<tr>
<td>Trunk</td>
<td>22%</td>
</tr>
<tr>
<td>Hand</td>
<td>29%</td>
</tr>
<tr>
<td>Upper Limb &amp; Shoulders</td>
<td>9%</td>
</tr>
<tr>
<td>Other</td>
<td>18%</td>
</tr>
</tbody>
</table>

This triangle has been made into an inferential statistic such that characteristics at the top of the triangle – that is, high level damage – are inferred from the lower levels. The reader should be aware that the pattern of multiple fatalities is different from the pattern of single fatalities; is different from the pattern of non-fatal permanent disabilities and is different from the pattern of temporary and minor damage. For example, within the coal mining industry “fires, flooding and explosion” are most highly represented in multiple fatalities, whereas single “at work” fatalities involve “gravitational energy - falls of objects” and “vehicular energy” –vehicle to pedestrian...
strikes. When one examines some of the previous data given for less than five days off work, the high
involvement of eyes and hands become apparent. However, in the realm of non-fatal permanent disability it is
very seldom that eyes and hands are involved. It is predominantly the torso.

There is a mythology that site incident databases of reported near misses yield the necessary insight; however this
is contrary to the author’s experience. The near miss reports (by potential damaging energy) for an open cut
metalliferous mine during 2002, are illustrated in Table 12. This table shows the sensitivity to those energies
associated with fatality, i.e. vehicles and gravitational energy (falls from height) but does not show sensitivity to
the most common sources of non-fatal disability, that is, human energy, vehicular energy - jolting and jarring,
gravitational energy - falls to the same level, loss of grip at heel strike and falls while descending equipment
(fixed and mobile).

Table 12 Near Miss Reports by Potential Damaging Energy – Open Cut Metalliferous - 2002

<table>
<thead>
<tr>
<th>Potential Damaging Energy</th>
<th>Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle/Environment</td>
<td>8</td>
</tr>
<tr>
<td>Vehicle/Animal</td>
<td>8</td>
</tr>
<tr>
<td>Vehicle/Vehicle</td>
<td>3</td>
</tr>
<tr>
<td>Object</td>
<td>6</td>
</tr>
<tr>
<td>Gravitational Fall from height</td>
<td>5</td>
</tr>
<tr>
<td>Falling Object</td>
<td>5</td>
</tr>
<tr>
<td>Electrical</td>
<td>3</td>
</tr>
<tr>
<td>Machinery</td>
<td>3</td>
</tr>
<tr>
<td>Chemical</td>
<td>2</td>
</tr>
<tr>
<td>Other pressure</td>
<td>2</td>
</tr>
<tr>
<td>Thermal</td>
<td>1</td>
</tr>
<tr>
<td>Susceptible part</td>
<td>1</td>
</tr>
</tbody>
</table>

If the proposition is accepted that vehicular energy, gravitational energy and human energy are highly represented
in non-fatal permanent disability then one would expect such incidents involving “near misses” would be reported
into an incident database. However when the ratio of personal damage incidents to all recorded incidents for the
energy types which permanently damage people are examined, it can be observed that there is an apparent
sensitisation to vehicle related incidents but a significant desensitisation to human and gravitational incidents.
Table 13 illustrates.

Table 13 Ratio of Damaging Incident to All Incidents by Energy Type

<table>
<thead>
<tr>
<th>Energy Type</th>
<th>Personal Damage Incidents</th>
<th>All Recorded Incidents</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobile Equipment</td>
<td>54</td>
<td>366</td>
<td>1:7</td>
</tr>
<tr>
<td>Gravitational Energy (Fall of People, Fall of Objects)</td>
<td>106</td>
<td>235</td>
<td>1:2</td>
</tr>
<tr>
<td>Human Energy</td>
<td>366</td>
<td>389</td>
<td>1:1</td>
</tr>
</tbody>
</table>

When one further examines the reported incidents for vehicular energy in the above table they do not capture the
“jolting and jarring” experience but capture those incidents associated with vehicle loss of control situations.
Therefore, an organization can only be disappointed if they believe their site incident database will yield the
necessary insights. There are papers which suggest the most comprehensive source of information is found in the
experience and knowledge base of the people who complete the work. People do not associate and make the
linkage between their experience and the likelihood of non-fatal permanent disability, for example, a miner being
jolted and jarred in an underground transport vehicle is simply that – an uncomfortable experience which may
create pain and bantering between the driver and passenger. A person completing a heavy lifting task and not
experiencing pain is simply completing a “mongrel job”. A person descending the boot end of an underground
longwall and jumping 800 mm to the ground does not associate the situation with the potential for non-fatal
permanent disability.
HARVESTING THE EXPERIENCE OF THE PEOPLE

Harvesting of information is achieved by interviewing individuals or small groups using a framework of focussed questions which cause a person to have a definite frame of reference in organising their thinking patterns during the interviewing process. For example, imagine the difference in the information that can be obtained if one were to ask the following questions:

**Question:** Where do you think you are most likely to be injured on this site?

**Versus the following options:**

1. Please describe to me, tasks which you complete that you would subjectively describe as heavy or very heavy lifting/pushing or pulling tasks?
2. Please describe to me where you work at a height where, if you were to overbalance or fall to accommodate some critical information at an appropriate time, you could fall 1 m or more and be seriously injured.
3. Please describe to me surfaces about your workplace where your foot has slipped forward rapidly as you were walking and/or working.

The difference in questioning is very simple, but profound in terms of the results it produces. It is possible to develop a set of focussing questions against those energy types known to damage people e.g. Human Energy, Gravitational Energy, Vehicular Energy, Electrical Energy, or Chemical Energy. There are a number of recorded techniques in the literature for harvesting the “store of” information within a workforce. That which has been recorded over the longest duration is a technique known as Critical Incident Recall (in excess of 90 years). Focussed Recall, and Perception Analysis are other techniques. The process involves the following three steps.

1. Problem identification of potentially permanent damage based on a workforce’s experience set against a framework of focussing questions.
2. Prioritisation of problems followed by analysis using an appropriate model to generate solutions.
3. Implementation of solutions followed by an audit to determine effectiveness.

A significant factor in this predictive process is the correct combination of site knowledge combined with outside expertise. That expertise can either be employed into an organization or is transferable into the organization so that the organization itself combines expertise with site knowledge. Expertise should be inherently organised and communicable – experience is not. Therefore, the combination of expertise and experience has the potential to document the detail of experience against the generalised pattern of permanent personal damage.

One of the fundamental principles underlying the harvesting of experience is as follows:

<table>
<thead>
<tr>
<th>PRINCIPLES:</th>
</tr>
</thead>
<tbody>
<tr>
<td>• 10 PEOPLE EACH WITH 15 YEARS OF EXPERIENCE</td>
</tr>
<tr>
<td>• 150 YEARS OF POSSIBLE EXPOSURE</td>
</tr>
<tr>
<td>• POSSIBLE LIKELIHOOD OF ONE PERMANENT DISABILITY CASE</td>
</tr>
</tbody>
</table>

Harvesting the information contained in the store of knowledge of a workforce is a problem identification process. However, it is necessary to have appropriate goals when embarking upon such a process. A summary of appropriate goals could be as follows:

**Goal 1** Direct 80% of safety effort towards the prediction and management of future potential permanent personal damage.

**Goal 2** Maximise understanding of future potential non fatal permanent damage.

This requires that those who are allocated to problem solving be prepared to challenge their own experience base as well as the industry norms with respect to how tasks are completed.

**Goal 3** Apply a multi-factorial model in understanding potential future damage as opposed to a single factorial model.

A single factorial model often uses ‘cause/effect’ terminology wherein an attempt is made to understand basic causes, root causes, and main causes. However the application of a model...
where understanding and insight is obtained through asking non-value, non-judgmental, non-emotive questions such as:

1. What did people do and what did people not do that could be essential in the propagation of damage?
2. What features of equipment are present and what features of equipment are absent that could be essential in the propagation of damage?

It is to be noted that the terms “safe” and “unsafe” are not being used in the previous two questions. These are value judgment terms and could result in rejection of information before it is recorded. Judgements with respect to “safe” and “unsafe” are different between cultures, different between and within organizations and different between and within individuals.

Expectation plays an important role in the processing of information with respect to future potential damage and in particular with respect to the selection of appropriate solutions. A common expectation is that 88% of accidents are caused by human error, 10% by machine design and 2% by Acts of God, i.e. the 88:10:2 rule. Not only is this statement scientifically nonsensical, it is also theologically nonsensical. The only correct statement that can be made is that in actual and potential incidents, behaviour factors, design factors and environmental factors were either present of absent in 100% of cases. The correct ratio is 100:100:100. Therefore, when harvesting experience, expect to observe contributions for people, equipment and the working environment.

Unfortunately it would appear that in the 88:10:2 ratio or some variation of it is still in favour and will hinder progress. The Queensland Mines and Quarries Safety Performance and Health Report 1st July 2000 - 30th June 2001 (2002) under equipment causal factors for the year 2000 indicates that no equipment factors were involved in 60% of cases, no environmental factors were involved in 60% of cases, no human factor involved, implying that human factors were involved in 12% of cases. This statement is simply a variation on the 88:10:2 theme and unfortunately helps to promote the mythologies that hinder progress.

If a person were to approach a study of tasks that had the potential to create future permanent personal damage with the following four major areas of control as their dominant information organisers, their expectation would have a significant influence on their final recommendation.

- Control Measure 1: the person was not adequately trained;
- Control Measure 2: the person was not following procedures;
- Control Measure 3: the procedures were inadequate;
- Control Measure 4: the person was not wearing appropriate personal protective equipment.

These categories of control measures are commonly observed on Incident Report Forms. They create an expectation with respect to control measures when incidents are being analysed. When considering the major potential damaging energy of Human Energy, it is common to find that statements are made with respect to the person not following correct procedures, not adequately trained, the training was not adequate. Having investigated many hundreds of Human Energy/simple lifting permanent damage cases, it is most frequently the case that well established scientific guidelines with respect to acceptable moments (load x distance) of lift are exceeded. The foregoing set of expectations with respect to training and procedures will not yield the necessary gains with respect to the management of Human, Gravitational and Vehicular Energies.

It is possible that the focus on people as a control measure at the point of task has plateaued in terms of its ability to influence future personal permanent damage and that a different and more effective hierarchy of controls has to be more widely applied.

**PATTERN OF POTENTIAL DAMAGE ARISING FROM FOCUSED INTERVIEWS – OAKY CREEK**

The pattern shown in Figure 4 arises from the group interviews for the Oaky Creek No. 1 Development Crew. The crew were interviewed in sessions of one and a half to two hours per group of five to seven people. The numbers in Figure 4 simply reflect the individual items for correction. The numbers do not reflect the number of people who raised the issue. Table 14 and Table 15 are a summary of items identified for correction under “human energy”, heavy lifting/overexertion for the long wall crew and development crews.
FIG. 4 - Oaky Creek Focussed Recall Taxonomy – Development Crew

Table 14  Items Identified For Correction by Longwall Crew – Human Energy – Heavy Lifting

<table>
<thead>
<tr>
<th>Item 2.1</th>
<th>Task Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1.1</td>
<td>Lifting Components on to Longwall</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Removing Belt Structure</td>
</tr>
<tr>
<td>2.1.3</td>
<td>Monorail Removal</td>
</tr>
<tr>
<td>2.1.4</td>
<td>Removing Pipes in the Tailgate</td>
</tr>
<tr>
<td>2.1.5</td>
<td>Lifting Heavy Coal/Stone</td>
</tr>
<tr>
<td>2.1.6</td>
<td>3.3kV Cable – Monorail to Transformer &amp; Transformer to Main Line</td>
</tr>
<tr>
<td>2.1.7</td>
<td>High Tension Plugs</td>
</tr>
<tr>
<td>2.1.8</td>
<td>Lifting Bretby Cable</td>
</tr>
<tr>
<td>2.1.9</td>
<td>Belt Spindles</td>
</tr>
<tr>
<td>2.1.10</td>
<td>Installing Dog Bones</td>
</tr>
<tr>
<td>2.1.11</td>
<td>Lifting Flights</td>
</tr>
</tbody>
</table>
Table 15  Items Identified For Correction by Development Crew – Human Energy – Heavy Lifting

<table>
<thead>
<tr>
<th>Item 10.1.1.1</th>
<th>Vent Tube Installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item 10.1.1.2</td>
<td>Vent Tubes into/out of Shuttle Cars</td>
</tr>
<tr>
<td>Item 10.1.1.3</td>
<td>Installation of Water/Air Pipes</td>
</tr>
<tr>
<td>Item 10.1.1.4</td>
<td>Supplying The Miner</td>
</tr>
<tr>
<td>Item 10.1.1.5</td>
<td>Moving/Extending Boot End Conveyor</td>
</tr>
<tr>
<td>Item 10.1.1.6</td>
<td>Hanging Miner Feeder Cable</td>
</tr>
<tr>
<td>Item 10.1.1.7</td>
<td>Installing Cable Bolts</td>
</tr>
<tr>
<td>Item 10.1.1.8</td>
<td>Carrying Drums of Oil</td>
</tr>
<tr>
<td>Item 10.1.1.9</td>
<td>Lifting DAC Cable Rolls</td>
</tr>
<tr>
<td>Item 10.1.1.10</td>
<td>Removal of Electrical Enclosure Barrier – JOY Continuous Miner</td>
</tr>
<tr>
<td>Item 10.1.1.11</td>
<td>Changing Shuttle Car/Loader Tyres</td>
</tr>
<tr>
<td>Item 10.1.1.12</td>
<td>High Tension Cable Plugs</td>
</tr>
<tr>
<td>Item 10.1.1.13</td>
<td>Lift Cylinder Change Out – Tail of Continuous Miner</td>
</tr>
<tr>
<td>Item 10.1.1.14</td>
<td>Removing Shuttle Car Tractor Motor</td>
</tr>
</tbody>
</table>

Figure 5 illustrates how the pattern of recalled experience correlates to the pattern of non fatal permanent damage for the industry.

FIG. 5 - Correlation – Oaky Creek Focus Recall to Underground Coal Mine Taxonomy
NSW-QLD 1990-1995
AN EXAMPLE

The following two items were captured in respect to monorail removal (human energy/heavy lifting): head impact when jumping off the monorail platform (human energy/impact) and the potential for falling (gravitational energy/fall of person). They are described to show how the recalled and reported information is presented to the client.

**Item 2.1.3 Monorail Removal**

**Frequency of Exposure:** Several times per shift  
**Potential Consequence:** Lumbar/cervical disc damage  
**Damaging Energy Type:** Human Energy – heavy lifting/pushing/pulling

Monorails must be removed as the Longwall retreats. The Monorail is used for the support of hoses that deliver energy, e.g. electricity and hydraulic oil etc, to the Longwall. One person completes the task. Work is completed at shoulder height and above. The workperson lifts one end of the Monorail off the supporting chain and then lowers that end of the Monorail until the other end can be disengaged from the mating connection. The item is then thrown into the Monorail pod, to do this task the person stands on the platform. The platform is accessed by a 700mm step up from an immediately adjacent and lower platform. That platform is also longitudinally displaced from the other.

There is potential for impact through head strike on the roof if the platform is too high relative to the roof. The task is described as requiring “moderate strength” if the platform is so located relative to the underside of the Monorail that the person can adopt a satisfactory posture. However, due to the lateral displacement of boot end equipment within the Main Gate, the person can find that the Monorail is significantly horizontally displaced to the side of the platform such that they have to reach beyond the platform.

The task is described as requiring moderate strength when the person can optimally position himself relative to the Monorail which is 1.83 m long and weights 32 kg. It is necessary to understand that the handrails are removable and the platform is able to pivot to a stored position. This allows for removal of the pod by an Eimco when the pod becomes adjacent to a Cut Through.

The ergonomics of the task, as described, were less than optimum. Posture would predictably overload the musculoskeletal structure.

**Recommendation 1.** It is recommended that this platform be made height adjustable and laterally adjustable via foot control and that such adjustment be completed hydraulically. The adjustable platform could be easily detached from the pod adjacent to the Cut through by having two quick connect hydraulic couplings and it would be possible to leave the platform in a stored position so that it does not intrude into the Main Gate more than currently occurs. It is necessary for this platform to be equipped with a foldable 75 degree ladder to allow for a transition to the immediately adjacent lower platform.

**Item 2.3.3 Jumping Off Monorail Platform**

**Frequency of Exposure:** Daily  
**Potential Consequence:** Non-fatal permanent damage – musculoskeletal  
**Damaging Energy Type:** Human energy – impact, Gravitational Energy – fall of person

The monorail platform is attached to the monorail pod. This platform has a height of 1500 mm above floor level and 700 mm above the immediate adjacent platform. The transition from the monorail elevated platform to the lower platform is hazardous. The task has been observed and people either attempt to step down, jump down, or alternatively, seek a very insecure foot hold with their left foot on a bracket on the BSL while boots are muddy and the surface is wet and contaminated. There is a high risk of people slipping on making this transition. The transition is made every 2 m of retreat i.e. up to five times per shift.

On the current platform the handrails are removable and the platform is hinged to allow an Eimco to remove the pod.

This item has been discussed under Section 2.1.3. If not other changes are made with respect to relieving the musculoskeletal stresses in monorail removal, it is necessary to improve the quality of the transition from this elevated platform to the lower platform.
**Recommendation 1:** Incorporate this item into Item 2.1.3 i.e. an upgraded platform associated with monorail removal. An interim solutions is to place a set of transition steps from the upper to the lower platform.

These items were subsequently audited in November 2002 and the following is a description of the audit results.

**Item 2.1.3  Monorail Removal**

**Overall Audit Assessment:**

<table>
<thead>
<tr>
<th>POOR</th>
<th>FAIR</th>
<th>GOOD</th>
<th>EXCELLENT</th>
</tr>
</thead>
</table>

**Original Recommendations:**

It is recommended that this platform be made height adjustable and laterally adjustable via foot control and that such adjustment be completed hydraulically. The adjustable platform could be easily detached from the pod adjacent to the Cut through by having two quick connect hydraulic couplings and it would be possible to leave the platform in a stored position so that it does not intrude into the Main Gate more than currently occurs. It is necessary for this platform to be equipped with a foldable 75 degree ladder to allow for a transition to the immediately adjacent lower platform.

**Observation and Comments:**

There has been significant work completed in this area. There are now three pods for the receipt of monorails as opposed to previously one larger pod. The beam is handled less often e.g. twice instead of six times, and there is a hydraulic platform which is hydraulically retractable. The weight of the monorail beam remains the same at 32 kg, it is recognised that there can be lifting and twisting issues involved with one person moving the beam and hence the recommendation relates to providing two people to release and place a monorail beam.

**Recommendations arising:**

It is recommended that the task of removing monorail beams and placing them into the pods be completed by two people.

Each of the problems identified in the focussed recall document were systematically audited and additional recommendations made as required. It is considered that there has been a reduction in the potential for non-fatal permanent disability for the Oaky North and Development Crews as a consequence of embarking on the work and implementing the change. Work is still not yet complete, however the authors consider that the “process” is more important than an outcome measure. The difficulty with non-fatal permanent disability is that there are “so many” in the life of a country or an industry but “so few” in the life of an organization. Therefore measuring non-fatal permanent disability using some annual incidence rate with respect to an individual site does not hold a lot of relevance. It is far better to implement processes strongly focused at identifying exposures and then have appropriate models for analysing and implementing change.

**CONCLUSION**

An has been made to quantify the size and nature of the industrial personal damage problem in terms of the numbers of people involved and the predominant damaging energy types. Reference has been made to the likelihood of non-fatal permanent disability in industry generally and for the coal industry specifically.

The incident rate for non-fatal permanent disability in the coal industry is considered very high at typical 1:200 person years worked. No organization has set a guide as to what is “acceptable likelihood”. The author would suggest at least one in 10,000 person years worked which would be a fifty-fold improvement. This would reduce the number of people currently permanently disabled from work in this country from 50 000 per year to 1 000 per year.

The enormous value of the knowledge of the workforce has been expounded. The collection and recording of such knowledge and experience requires a very structured approach using a set of focusing questions that are established against a backdrop of that which is known to permanently damage people. In particular, the questions must contain reference to human energy (heavy lifting/pushing/pulling tasks), gravitational energy, falls of people, falls of objects, vehicular energy (in particular jolting and jarring). Having harvested the store of experience in the workforce it is necessary to have the appropriate goals and expectations in response to handling that information.
It is suggested that the coal industry has a high likelihood of non-fatal permanent disability. It is suggested that there is currently not an appropriate industry focus directed toward that level of personal damage in the industry. It is suggested that the industry needs to promote and implement processes which can clearly identify the potential for non-fatal permanent disability and effectively manage the future likelihood of that potential consequence.

REFERENCES

RESPIRABLE DUST RESULTS FROM NSW LONGWALL MINES

Ken Cram

ABSTRACT: An outline of airborne dust sampling methodology, instrumentation and exposure standards in the New South Wales coalmining industry are presented combined with the results of dust monitoring of the mining industry workforce with particular emphasis on longwalls. The overall improvements to workforce exposure levels and the systems and techniques which led to these improvements are dealt with, as are respirable quartz levels, together with sources and difficulties of compliance with the Coal Mines Regulation Act (CMRA) 1982 requirements. The non-punitive nature of the regulations combined with the mutual co-operation between management and unions in the interpretation and use of monitoring program results, to achieve overall improvements in airborne dust levels throughout the industry is examined. Results are presented covering long wall, continuous miner and open cut / surface operations. Periodic health screening and results of epidemiology studies of the workforce by Coal Services Health (formerly the Joint Coal Board) indicate adherence to current maximum exposure levels is sufficient to maintain a healthy industry workforce.

INTRODUCTION

The Joint Coal Board was originally constituted under an arrangement between the Governor-General of Australia and the Governor of New South Wales made pursuant to the provisions of the Coal Industry Act, 1946 (Commonwealth) and the Coal Industry Act 1946 (NSW). In 2001 the Commonwealth withdrew from the Joint Coal Board and those Acts were repealed. As a replacement for the Joint Coal Board under the Coal Industry Act (NSW Govt 2001) a corporation was formed, Coal Services Pty Limited to oversee occupational health and welfare in the NSW coal industry. Coal Services is owned equally by the New South Wales Mineral Council and the CFMEU (mine workers union). The powers and functions of the corporation are stated in the provisions of the Act and include the responsibility to monitor respirable dust in NSW coal mines.

Coal Services Health operates the dust sampling program, which is an occupational hygiene service and is complementary to the other health services provided which include mine workers biological monitoring from chest x-ray examination and lung function tests. Since July 1994 the dust sampling service has been on a fee-for-service basis.

HEALTH RISK

The health risk to mine workers has long been acknowledged as being related to prolonged exposure to high concentrations of respirable coal dust which can lead to pneumoconiosis, and when mining high quartz content material, silicosis. Coal mining has historically been associated with the occurrence of disabling chest diseases.

The International Labour Organization (ILO) Classification System, the international standard, is the system used by Coal Services Health to grade pneumoconiosis on chest radiographs of coal mineworkers. Under this system, there are four major categories used to grade the severity of pneumoconiosis. Essentially, category 0 is the normal state (no pneumoconiosis), category 1 is mild pneumoconiosis, category 2 is moderate pneumoconiosis, and category 3 is severe pneumoconiosis. It is generally agreed by clinicians, that symptoms of pneumoconiosis are not experienced until category 2 is reached. At category 1, most individuals would be unaware of the presence of early pneumoconiosis, and would not normally be restricted in work or leisure activities.

When the Joint Coal Board was established in 1948 pneumoconiosis was prevalent among coal miners (16% all categories, 4.5% category 2 or worse). Today, the prevalence is so low that no new cases of pneumoconiosis have been detected in the last 10 years. The incidence of pneumoconiosis in NSW is among the lowest in the world. For the last 5 years the rates of pneumoconiosis in NSW continues to be less than 0.5% (Joint Coal Board 2001)

1 Coal Services Health
STANDING COMMITTEE ON DUST RESEARCH AND CONTROL

The Joint Coal Board since its inception actively pursued the eradication of dust related diseases among coal miners. In 1954 a Standing Committee on Dust Research and Control was established to provide expert advice to the NSW coal industry on respirable dust issues. The committee was constituted from representatives of the colliery proprietors, mining unions, government departments and the Coal Services Health division. The committee was instrumental in the introduction of the gravimetric sampling method and setting the current exposure standards. Coal Services Pty Limited is continuing those same objectives in 2002.

The Committee meets bi-monthly to review results of Coal Services Health monitoring programs and evaluate and exchange information on technologies, innovations and problems in the industry related to respirable dust. The role of the committee in 2002 remains fundamental to the promotion of improved health standards for coal industry workers.

DUST MONITORING SERVICE

The Coal Services Health (formerly the Joint Coal Board and JCB Health) dust monitoring service is quality accredited and has been the sole organization involved with personal dust monitoring in the NSW coal industry since the current regulations (CMRA, 1982) were gazetted in March 1984. The service has the total support and acceptance of both management and unions.

The specified limit for respirable dust other than quartz-containing dust, is 3mg of respirable dust/m$^3$ of air sampled. The specified limit for quartz-containing dust is 0.15mg of respirable quartz/m$^3$ of air sampled (CMRA, 1982). The details are in the attachment (Appendix A).

The frequency of sampling, places and persons to be sampled in each part of a mine are specified in the table (Appendix B). In NSW sample collection commences at the time of leaving the crib room at the start of the shift and ceases on arrival at the crib room at the end of the shift. The sampling period, if practicable should be not less than five hours (CMRA, 1982).

While it is the responsibility of mine management to meet the frequency of sampling required by the CMRA the Coal Services Health monitoring programs are structured in such a manner that management’s obligations are fulfilled where possible.

The integrity of results is guaranteed by a Coal Services Health employee present in the workplace during the sampling shift recording such information as ventilation quantities, blocked sprays, operator location, water pressures or anything which may affect results. Results are used solely to identify problem areas which may exist and are not used at any time for punitive measures. Where areas of high dust concentrations are found efforts are directed to these areas in order to rectify the problems. These efforts in many cases involve Management, Union and Coal Services Health initiatives.

Results of the sampling are forwarded to the colliery manager, senior inspector of coal mines, united mineworkers district check inspector and included in the Coal Services Health dust database.

If the result of any sample exceeds the specified limit a re-sample must be taken within seven working days in similar circumstances to those existing when the sample was collected. If the resample still exceeds the specified limit the district inspector of coal mines may, in writing direct the colliery manager to carry out additional procedures to reduce the concentration of airborne dust (NSW Govt. 1999).

During the period 1984-2001 the number of underground mines in NSW reduced from 67 to 32, mainly with the closure of non-longwall mines. Open cut mines increased in that period from 18 to 24 mines. Total raw coal production in the period increased from 68.3 million tonnes in 1984 to 142.9 million tonnes in 2001. Underground production increased from 42.2 million tonnes to 54.6 million tonnes and open cut production substantially increased from 30.7 million tonnes to 88.4 million tonnes during the same period (Coal Services 2002).
COAL SERVICES HEALTH DUST DATA BASE RESULTS

By the end of 2001, after nearly 18 years of sampling, over 46,000 personal dust samples (including re-samples which is the worse case scenario) had been collected from over 9,000 mining locations. Sampling locations were 30% longwall faces, 62% underground other than longwall (mainly continuous miner panels) and 8% open cut/washeries. From 1984 to 1997 the sampling location mix was 65% underground other than longwall and 5% open cut/washeries, increased sampling at open cuts in the last 4 years has seen that % change. An average of over 2,500 personal dust samples at over 500 mining locations were collected per annum from 1984 to 2001.

Analysis of the data is based upon the results obtained during the standard frequency sampling and re-samples. The results of re-samples taken for the requirements of CMRA have been included in the data analysis in Table 1

Table 1 Respirable Dust Results (Including Re-Samples) 1984 - 2001

<table>
<thead>
<tr>
<th>Mining Method</th>
<th>Number of Personal Samples (Including Re-Samples)</th>
<th>Number &gt; 3mg/m³</th>
<th>Percentage Exceeding Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longwall Faces</td>
<td>14170</td>
<td>1002</td>
<td>7.1</td>
</tr>
<tr>
<td>Other Underground</td>
<td>29040</td>
<td>468</td>
<td>1.6</td>
</tr>
<tr>
<td>Open Cut/Washeries</td>
<td>2790</td>
<td>21</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Examination of the dust results by mining method gives a clearer understanding of the situation shown in Table 1

- Longwall operations - over 14,000 samples and 7.1% exceeded the limit.
- Other underground (mainly continuous miner panels) - 29,000 samples and 1.6% exceed the limit.
- Opencut/washeries - 2,790 samples and only 0.8% exceed the limit.

This clearly shows that the area of main concern has been the results from the longwall faces.

LONGWALL RESPIRABLE DUST RESULTS

In 1984 there were 12 longwall faces, which progressively rose to 25 faces by 1997 but had reduced to 21 by 2001. Longwall samples over the 18 year period resulted in over 7% of the samples exceeding the 3mg/m³ level. Details of the results are shown in Table 2 where significant improvement has been achieved in the results over the 18 year period. During the 1980s the percentage of results exceeding the limit peaked at over 18%. From 1990 substantial initiatives by coal companies achieved the present situation where only 6% of results exceed the limit but longwall mining results still remain the main area of concern. The overall trend during the period has been a reduction in the percentage of samples exceeding the limit. There had been a slight deterioration in the late1990’s and this was attributed to a few particular longwalls where there were operational problems. These are being addressed and the trend in results exceeding the limit is expected to continue going down in the future.

It should be noted that two significant changes occurred in the 1984-2001 period. Firstly the number of longwall faces had doubled and the average shift longwall face output increased by more than 100%. During 2001-02 NSW longwall production was from 22 mines with the coalfield details as follows:

<table>
<thead>
<tr>
<th>Coalfield</th>
<th>Production</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hunter Coalfield</td>
<td>10.8 million raw tonnes from 5 faces</td>
</tr>
<tr>
<td>Newcastle Coalfield</td>
<td>8.6 million raw tonnes from 6 faces</td>
</tr>
<tr>
<td>Western Coalfield</td>
<td>10.0 million raw tonnes from 4 faces</td>
</tr>
<tr>
<td>Southern Coalfield</td>
<td>10.9 million raw tonnes from 7 faces</td>
</tr>
<tr>
<td>Total NSW Coalfields</td>
<td>40.3 million raw tonnes from 22 faces</td>
</tr>
</tbody>
</table>

Respirable dust monitoring results for all NSW longwalls after 18 years monitoring has achieved 14,170 personal samples with just over 1,002 (7.1% failures) exceeding the prescribed limit. Which longwalls or coalfields are contributors to these failures?
Western coalfield mines are not a significant contributor 1,500 personal samples and only 52 failures (3.5%). In 1987 and 1994 a few dust problems affected results at Clarence and Ulan. In 2001 there were some high dust results on a Lithgow seam longwall mine which are being addressed.

In Southern coalfield mines, which were the forerunner for longwall mining, 5,930 personal samples produced 465 failures (7.8%). The NSW dust failure trend has traditionally followed the performance of the southern coalfield mines which had generally half the longwall samples and half the failures. There had been a concerted effort in the late 1980’s to drive the high 30% of failures down to the present level of 3 – 5%. In the last 5 years the southern coalfield results have been better than all the NSW percentage failures, although the number of southern faces has reduced by 30%.

Northern coalfield (Newcastle) longwall mines had 4,770 personal samples and 276 failures (5.8%). During the 1980’s results were better than the all NSW percentage failures, from the mid 1990’s the results have not been as impressive and the situation up to year 2000 was worse than the combined results of the NSW longwall faces. The main contributor during those years had been the new faces in the West Borehole and Great northern seams. Combined efforts were concentrated to reduce the personal exposure on the longwall faces to the present level of 5 – 6 % of results failing. The number of northern faces has been reasonably static for the last 10 years.

Hunter coalfield (Singleton) longwall mines have had 1,960 personal samples and 209 failures (10.7%). The number of faces has doubled in the last 10 years and since the mid 1990’s the results have seen the number of failures (8 – 10%) far in excess of the NSW number of failures, around 6%. Initially the main contributors were faces in the Whybrow and Pikes Gully seams. From the mid 1990’s the Wynn seam at Dartbrook Colliery has been difficult to longwall mine and maintain dust levels below the prescribed limit. All the Hunter coalfield longwall operations have been continually trying to improve dust suppression measures and operating procedures to reduce the face operators dust exposure levels.

Longwall dust suppression has been very successful in the following areas:

- sealing the covers on the BSL and enclosing the BSL discharge on the belt to reduce intake contamination
- homotropol ventilation has been very successful in allowing clean uncontaminated air onto the longwall face
- water infusion in the Bulli seam utilising in-seam gas drainage holes has been reasonably successful in putting some moisture back into the seam
- operator location with emphasis on face operating procedures has been a major contributor to the improved longwall face dust results. The ‘Hund’ instrument has been an excellent tool to highlight areas of high dust levels and indicate the best location for face operators
- shearer initiation of chocks (shields) has also moved people from the return side of the shearer.

RESPIRABLE QUARTZ RESULTS

Analysis of the quartz data is based upon the results obtained during the standard frequency sampling and re-samples. The results of re-samples taken for the requirements of CMRA have been included in the data analysis.

Of the 46,000 personal samples taken around 4,500 were sent for quartz analysis. The samples sent for analysis were from those locations where the mining practice or material was expected to contain high quartz containing dust and where the sample failed the respirable dust limit. In the case of longwall samples 20% were sent for quartz analysis. Samples from longwalls sent for quartz analysis had a 1 in 3 chance of exceeding the specified limit of 0.15mg of respirable quartz/m³. Those samples exceeding the specified limit for different mining methods for the period 1984 - 2001 are detailed in Table 3.
TABLE 3  
RESPIRABLE QUARTZ RESULTS (INCLUDING RE-SAMPLES) 1984 – 2001

<table>
<thead>
<tr>
<th>Mining Method</th>
<th>Number Of Personal Samples (including re-samples)</th>
<th>Number &gt; 0.15mg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Opencut/Washeries</td>
<td>2790</td>
<td>38</td>
</tr>
<tr>
<td>Longwall Faces</td>
<td>14 170</td>
<td>536</td>
</tr>
<tr>
<td>Other Underground</td>
<td>29040</td>
<td>382</td>
</tr>
</tbody>
</table>

LONGWALL QUARTZ RESULTS

High quartz has resulted from longwall mining through stone rolls, faults and dykes or when cutting roof and floor. Some coal seams in the Hunter coalfield are high in quartz and are proving difficult to maintain below the CMRA maximum allowable limit of 0.15mg/m³. Examining the quartz results in a worst case scenario including re-samples, 536 failures 3.8% of 14,170 personal samples.

The western coalfield had 21 failures and this is only 1.4% of the 1,510 samples. Overall there have been only a few problems over the last 10 years. In the last 2 years there have been high results at a Lithgow seam mine cutting roof stone in sections of the longwall block. The southern coalfield had only 31 failures, 0.5% from 5,930 samples. This coalfield and the western coalfield traditionally only have high quartz results when longwall faces are cutting roof, stone rolls or floor stone.

Quartz results in the northern coalfield (Newcastle) showed 302 failures, 6.3% from 4770 samples. This coalfield from 1985 – 95 was far above elsewhere in NSW and a main contributor to the high trend of failures in NSW. The Great Northern seam longwalls had high quartz results during this period, overall in the last 5 years the situation has improved. The Hunter coalfield had 182 failures from 1960 samples, which is over 9.3%. This coalfield has impacted greatly on the quartz results trend in NSW. The longwalls in the Whybrow seam have been the area of main concern, high quartz content in the seam and a need to operate at half the respirable dust levels to avoid quartz failures. The need to cut substantial amounts of roof on the longwall faces in the Pikes Gully and Liddell seams is going to be an area that needs to be addressed.

PERSONAL PROTECTION

Where dust exposure cannot be maintained below the specified limit, personal protection should be introduced but due to the protection factor of respirators being relative to facial fit and wear time (uncontrolled factors), they should only be used as a last line of defence and must not take the place of prevention or dust suppression techniques.

COAL SERVICES ORDER 40 - ABATEMENT OF DUST ON LONGWALLS

Another initiative of the Joint Coal Board and continued by Coal Services by its role in the area of airborne dust was the issue of Order 40 on 5 July 1990. This order requires the manager or owner of any mine operating by longwall or shortwall mining methods to obtain Coal Services approval prior to the commencement of production in any longwall or shortwall block. Results of dust samples from previous longwalls are examined prior to approval. Most approvals granted are subject to some form of imposed conditions.

The advent of Order 40 appears to have created a more positive and co-operative attitude towards dust control measures by both management and unions.
CONCLUSIONS

Results of the Coal Services Health Dust Monitoring programs combined with epidemiology studies indicate that adherence to the current maximum exposure standards outlined earlier is sufficient to maintain a healthy industry workforce.

Even though occupational lung diseases are currently well controlled in the New South Wales industry, it is essential that face management is vigilant to ensure that longwall machinemen adhere to face operating procedures, to limit dust exposure and that dust suppression equipment is maintained through engineering maintenance programs.

In mines operating in seams with high levels of inherent quartz and where there is a need to cut roof stone, it is necessary for operators to achieve lower than required dust levels in order to meet the specified levels of respirable quartz. Similarly additional dust suppression techniques may be required in development panels where conditions are such, that stone roof or floor must be continually mined.

Although recent annual reports from Coal Services former organization the Joint Coal Board have been indicating prevalence rates of pneumoconiosis in the NSW coal industry of less than 0.5%, respirable dust control management plans should still be a high priority.

Finally it is important that the industry does not become too complacent regarding the pneumoconiosis risk, particularly as memories of early miners disabled with chronic chest disease fade from the memories of the current workforce.

REFERENCES:

NSW Govt. 1982 Coal Mines Regulation Act 1982 No 67. NSW Govt. Printer: Sydney
NSW Govt. 1999 Coal Mines (Underground) Regulation 1999. NSW Govt. Printer: Sydney
NSW Govt. 2001 Coal Industry Act 2001. NSW Govt. Printer: Sydney
### Table 2 All Respirable Dust Results (Including Re-Samples) New South Wales Coal Mine 1984-2001

<table>
<thead>
<tr>
<th>Years</th>
<th>Underground Longwall Face</th>
<th>Other Underground</th>
<th>OpenCut and Washeries</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number of Personal Samples (including re-samples)</td>
<td>Number &gt;3mg/m³</td>
<td>Percentage exceeding limit</td>
</tr>
<tr>
<td>1984</td>
<td>238</td>
<td>40</td>
<td>16.8</td>
</tr>
<tr>
<td>1985</td>
<td>340</td>
<td>32</td>
<td>9.4</td>
</tr>
<tr>
<td>1986</td>
<td>307</td>
<td>38</td>
<td>12.4</td>
</tr>
<tr>
<td>1987</td>
<td>592</td>
<td>102</td>
<td>17.2</td>
</tr>
<tr>
<td>1988</td>
<td>553</td>
<td>118</td>
<td>21.3</td>
</tr>
<tr>
<td>1989</td>
<td>426</td>
<td>64</td>
<td>15.0</td>
</tr>
<tr>
<td>1990</td>
<td>645</td>
<td>61</td>
<td>9.5</td>
</tr>
<tr>
<td>1991</td>
<td>1112</td>
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<td>5.5</td>
</tr>
<tr>
<td>1992</td>
<td>1275</td>
<td>60</td>
<td>4.7</td>
</tr>
<tr>
<td>1993</td>
<td>1198</td>
<td>37</td>
<td>3.1</td>
</tr>
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<td>1994</td>
<td>1078</td>
<td>31</td>
<td>2.9</td>
</tr>
<tr>
<td>1995</td>
<td>922</td>
<td>48</td>
<td>5.2</td>
</tr>
<tr>
<td>1996</td>
<td>981</td>
<td>53</td>
<td>5.6</td>
</tr>
<tr>
<td>1997</td>
<td>1010</td>
<td>39</td>
<td>3.9</td>
</tr>
<tr>
<td>1998</td>
<td>1079</td>
<td>53</td>
<td>4.9</td>
</tr>
<tr>
<td>1999</td>
<td>866</td>
<td>59</td>
<td>6.8</td>
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<tr>
<td>2000</td>
<td>727</td>
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<td>7.8</td>
</tr>
<tr>
<td>2001</td>
<td>817</td>
<td>49</td>
<td>6.0</td>
</tr>
</tbody>
</table>
APPENDIX A

9154 OFFICIAL NOTICES 24 September 1999

COAL MINES REGULATION ACT 1982
COAL MINES (UNDERGROUND) REGULATION 1999
COAL MINES (OPEN CUT) REGULATION 1999

File No. C99/0691
Date: 1 September 1999

SPECIFIED LIMITS FOR AIRBORNE DUST

FOR the purposes of clause 161 of the Coal Mines (Underground) Regulation 1999 and clause 29 of the Coal Mines (Open Cut) Regulation 1999, (definition of ‘specified limit’), it is hereby notified that the limit specified in respect of certain types of dust is as follows:

Specified Limit for Quartz-Containing Dust:
The specified limit for quartz-containing dust is 0.15 milligrams of respirable quartz per cubic metre of air sampled.

Specified Limit for Respirable Dust (other than quartz-containing dust):
The specified limit for respirable dust (other than quartz-containing dust) is 3 milligrams of respirable dust per cubic metre of air sampled.

Definitions:
In this notice:

- “quartz-containing dust” means dust which contains five per cent or more by mass of respirable quartz
- “respirable dust” has the same meaning as it has in Australian Standard 2985
- “respirable quartz” means the quartz present in respirable dust
- “breathing zone” has the same meaning as it has in Australian Standard 2985

PAUL THOMAS HEALEY,
CHIEF INSPECTOR OF COAL MINES.

NEW SOUTH WALES GOVERNMENT GAZETTE No. 111
APPENDIX B

TABLE OF LOCATIONS, FREQUENCIES AND PERSONS FOR SAMPLING

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Frequency of Sampling</td>
<td>Persons to be Sampled</td>
</tr>
<tr>
<td>(a) in each part of the mine where longwall mining is carried out.</td>
<td>each producing shift at intervals not exceeding six months.</td>
<td>samples to be collected from the breathing zone of at least five persons including, where possible: - a shearer-loader operator, - two powered support operators, - a deputy, and - one other person to be selected by the manager.</td>
</tr>
<tr>
<td>(b) in each part of the mine where a continuous mining machine operates.</td>
<td>each producing shift at intervals not exceeding twelve months.</td>
<td>samples to be collected from the breathing zone of at least five persons in each unit including, where possible: - a continuous miner driver, - a sideman or cable handler, - a shuttle car driver, - a deputy, and - a boot end attendant or other person to be selected by the manager.</td>
</tr>
<tr>
<td>(c) in any place in or about an underground mine other than those referred to in (a) or (b) above, but including crusher stations and wateriest</td>
<td>at intervals not exceeding twelve months.</td>
<td>samples to be collected from the breathing zone of at least one person.</td>
</tr>
<tr>
<td>(d) in any place in or about an open-cut mine where dust may be present.</td>
<td>at intervals not exceeding twelve months.</td>
<td>samples to be collected from the breathing zone of at least one person.</td>
</tr>
</tbody>
</table>

NOTE:

(1) Any further samples required by regulation will be additional to these prescribed frequencies.
(2) In the case of (c) or (d) the manager shall select those activities where workmen are likely to be exposed to airborne dust. Such selection shall be notified on a yearly basis to the District Inspector who may require additional activities to be sampled.
(3) Samples and analyses conducted by or for the Joint Coal Board may be used by the manager as part or the whole of the required number of samples to be collected for a given period.
(4) Persons sampled shall, as far as possible, remain at the same job for the duration of the test.
THE MEASUREMENT OF AIRFLOW THROUGH REGULATORS

Hsin Wei Wu 1, Stewart Gillies 1 and Tim Mayes 1

ABSTRACT: One very reliable approach to establishing air quantity through a ventilation branch is through measurement of differential pressure across an opening or regulator. Mathematical relationships are available to relate (with some qualifications) pressure drop and quantity through an orifice placed symmetrically in a round flow conduit. However these can only be used to approximate mine regulator behaviour due to variability in construction, questions of symmetry and leakage.

Efforts to characterize and/or mathematically model a number of types of operating mine regulators are described. Results can be used in the development of a computerized monitoring and simulation system to provide immediate or real time data on air behaviour within each branch within an underground mine ventilation network through linking of sensors to the ventilation network simulation software. This new approach to ventilation provides improved understanding of airflows through all mine sections.

INTRODUCTION

There is a trend world wide to remote or telemetric monitoring of mine atmosphere conditions. Suitably robust and if required, intrinsically safe instruments are available for measurement of, for instance, gas concentrations, air velocity and air pressure. These are often tied extensive mine monitoring and communication systems.

One approach to establishing air quantity through a ventilation branch is through measurement of differential pressure across an opening or regulator (Gillies et al, 2002). Mathematical relationships are available to relate pressure drop and quantity through a regulator orifice placed symmetrically in a round flow conduit. However these can, at best, only be used to approximate mine regulator behaviour due to:

- The irregularity of mine regulators in shape and symmetry and their positioning in normally roughly square or rectangular mine airways,
- Whether air has to change direction before passing through the regulator,
- The construction of the mine regulator opening which may incorporate, for instance, the operation of louvres, a sliding door, window or curtain or placement of drop boards, and
- Uncontrolled air leakage through the regulator or adjacent bulkhead.

Efforts to characterize or mathematically model a number of operating mine regulators have been studied. The information has been used in a computerized monitoring and simulation system to provide immediate or real time information on each branch within an underground mine ventilation network through linking of sensors to the ventilation network simulation software. The system measures airflow or air pressure changes in selected ventilation branches and simulates flow through all other branches. This new approach to ventilation provides improved understanding of airflows through all mine sections. Ventilation network analysis software has been developed to link real time information generated by mine ventilation monitoring sensors into the network program to undertake network simulations and allow interpretation of key system data and operational changes (Gillies et al, 2002).

Some of the steps involved in a research project with particular emphasis on the examination, characterisation and modelling of various types of mine ventilation regulators used in both coal and metalliferous mines are described.

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1 University of Queensland
THEORY OF REGULATORS

A regulator is an artificial resistance (in the form of shock loss) introduced into an airway to control airflow. When airways are arranged in parallel and a prescribed quantity of air is made to flow through each branch, then “controlled splitting” is utilised. The branch with the highest resistance initially is termed the “free split”. The free split may remain untouched or “free” of added impedance during the controlled splitting exercise. Other branches can have resistance added to them by the use of regulators that induce a shock loss of the required amount to deliver assigned airflow along all branches.

Types Of Regulators

Regulators placed in mine air circuits may vary from well engineered devices with a long life to temporary roughly constructed arrangements that achieve a practical purpose or “the job in hand”. Some of the more permanent devices take the following forms.

Drop board regulators
Drop board regulators are a popular form of variable resistance regulators. They can consist of two vertical steel rails placed on each side of the airway (usually against bulkhead pillars) into which large wooden or steel boards are slotted from the ground up. Installation and alteration can be very labour intensive. More boards in place result in a smaller air opening and consequent generation of a higher shock loss. Personnel or vehicular access through them is usually difficult.

Louvres
Louvres form a variable resistance regulator. They are usually made of steel and are similar to domestic window louvres. The shock loss is related to the angle at which the louvres are open.

Rubber flaps
Rubber flaps can be used where vehicle access is required through a regulator and a good seal is not required. The flaps are hung from the back or roof, usually from a beam, such that they overlap. Vehicles can pass through them without the driver stopping the vehicle or opening the flaps.

Fabric or flexible material
These regulators may consist of fabric of flexible material stiffened by steel bars. The material may be tied to the back and allowed to hang freely across the drift. In this case air pressure should ensure that the material forms a reasonable seal against adjacent bulkhead pillars. The height of the material can be adjusted to vary the required shock loss. Alternatively the stiffened material may form a “roller door” arrangement that can be fully opened for passage of vehicles or blast concussion. The roller door opening action may be motorised, sometimes through remote control, to allow easy setting to achieve a range of shock losses.

Ventilation doors
Ventilation doors allow passage of personnel, vehicles and materials. They can completely seal off an airway (solid doors) or partially seal by incorporating an opening often covered by a sliding panel.

Ventilation bulkheads
In cases where a small amount of air is required a hole may be placed in a bulkhead. A sliding door may be used to control flow through the opening.

Derivation Of Regulator Equation

A regulator can be described as a large thin plate installed in a fluid conduit with an orifice cut in it. When a difference in pressure exists between the two sides the plate, fluid flows through the orifice in the pattern shown in Figure 1. The fluid enters the orifice from all directions or the high pressure side on the low pressure side. It issues as a converging jet in line with the centre of the orifice. The jet converges to its smallest area at a distance of about half the orifice diameter (Le Roux, 1990). This area is called the “vena contracta” (A_c at Fig. 1). The ratio between vena contracta and orifice area is the “coefficient of contraction”; C_c which is (A_c/A_r, in Figure 1).
Fig. 12 - Airflow pattern through an orifice (after Burrows et al, 1989).

McElroy (1935) found that the $C_c$ value is a relation between the ratio of the orifice and airway cross sectional area, $N$ ($A_r/A$ in Figure 1), and $Z$, which is an empirical factor designated as the contraction factor, and is expressed as:

$$C_c = \sqrt{\frac{1}{Z -ZN^2 + N^2}}$$

Values of $Z$ vary according to the edge shape of the orifice. Table 1 shows the $Z$ factor for various constructions.

Table 3 Contraction factors (Hartman et al, 1997)

<table>
<thead>
<tr>
<th>Edge</th>
<th>Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>Formed</td>
<td>1.05</td>
</tr>
<tr>
<td>Rounded</td>
<td>1.50</td>
</tr>
<tr>
<td>Smooth</td>
<td>2.00</td>
</tr>
<tr>
<td>Square</td>
<td>2.50</td>
</tr>
<tr>
<td>Sharp</td>
<td>3.80</td>
</tr>
</tbody>
</table>

Since most regulators are square edged, a $Z$ value of 2.5 is most commonly used in calculating $C_c$. Bernoulli’s equation can be applied to both sides of the orifice as shown in Figure 1 in order to calculate the velocity of the air and hence the airflow quantity.

A correction must be made for the contraction of the jet at the vena contracta. Since the orifice is larger than the vena contracta, orifice velocity is lower than in the vena contracta. The velocity equated based on Bernoulli’s equations is the velocity at the vena contracta. Therefore, the velocity at the orifice can be obtained with the following equation:

$$V_2 = C_c \sqrt{\frac{2\Delta P}{\rho}} \left(\frac{1}{\sqrt{1-N^2}-1}\right)$$

(1)

where $C_c$ is the coefficient of contraction, as described before. Since airflow quantity through regulator $Q = V_2 A_r$, it follows that:

$$Q = C_c \sqrt{\frac{2\Delta P}{\rho}} \left(\frac{1}{\sqrt{1-N^2}}\right) A_r$$

(2)

where $A_r$ is orifice opening area in $m^2$. 
FIELD TESTS OF REGULATORS

Field tests on several types of regulators were conducted at various underground metalliferous and coal mines. Initially verification of air behaviour in flow through regulators was investigated. Parameters measured were airflow quantity and pressure drop across regulator. From pressure drop measurements, airflow quantity through regulators can be calculated with Equation 1. Results of this calculation can be compared with measured values and the reasons for significant differences investigated.

Drop Board Regulator Tests

The regulator tested initially was a typical drop board regulator, as shown in Figure 2. Results of these tests are summarized in Table 2.

Based on $\Delta P_s$ measured, predicted airflow quantity through the regulator, $Q$, was then calculated with Equation 1. Values of $Q$ were compared with the measured quantity, $Q_m$, as set down in Table 2 and Figure 3. It can be seen from both the table and figure that the measured quantity is consistently larger than predicted. There are several possible reasons as follows.

Error during measurement

It is common for operator induced errors to occur during mine drift measurement especially in small cross sectional airways. The authors experienced difficulty when measuring air velocity by continuous vane anemometer traversing because of limited space to move freely. Also the body of the underground person provides a significant obstacle to airflow.

<table>
<thead>
<tr>
<th>Condition</th>
<th>$\Delta P_s$</th>
<th>$Q_m$ m$^3$/s</th>
<th>$Q$ m$^3$/s</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully closed</td>
<td>163</td>
<td>2.05</td>
<td>0.00</td>
<td>n/a</td>
</tr>
<tr>
<td>1 board off</td>
<td>125</td>
<td>2.53</td>
<td>0.82</td>
<td>209.8</td>
</tr>
<tr>
<td>2 boards off</td>
<td>96</td>
<td>3.02</td>
<td>1.44</td>
<td>109.6</td>
</tr>
<tr>
<td>3 boards off</td>
<td>73</td>
<td>3.33</td>
<td>1.89</td>
<td>76.0</td>
</tr>
<tr>
<td>4 boards off</td>
<td>58</td>
<td>3.35</td>
<td>2.27</td>
<td>47.7</td>
</tr>
<tr>
<td>5 boards off</td>
<td>47</td>
<td>3.46</td>
<td>2.58</td>
<td>34.3</td>
</tr>
<tr>
<td>6 boards off</td>
<td>36</td>
<td>3.62</td>
<td>2.74</td>
<td>32.0</td>
</tr>
<tr>
<td>7 boards off</td>
<td>30</td>
<td>3.75</td>
<td>2.96</td>
<td>26.6</td>
</tr>
<tr>
<td>8 boards off</td>
<td>25</td>
<td>3.82</td>
<td>3.14</td>
<td>21.5</td>
</tr>
<tr>
<td>10 boards off</td>
<td>19</td>
<td>3.86</td>
<td>3.58</td>
<td>7.8</td>
</tr>
<tr>
<td>12 boards off</td>
<td>12</td>
<td>3.90</td>
<td>3.61</td>
<td>8.1</td>
</tr>
<tr>
<td>14 boards off</td>
<td>7</td>
<td>3.89</td>
<td>3.46</td>
<td>12.4</td>
</tr>
</tbody>
</table>
Non-symmetrical condition and shape
Equation 2 was derived based on a circular orifice in the middle of a regulator plate. The regulator opening under study is rectangular, has square edges and is located on the upper side and opening is rectangular not round leading to distorted air patterns. In addition the air turns through a sharp right angle before entering the regulator.

Leakage
Leakage occurs due to the presence of gaps between boards and between the regulator frame and the airway walls. The leakage quantity primarily depends on regulator construction and the differential pressure drop across the opening.

An approach is proposed to model the difference as air leakage since measurement error and the non-symmetrical condition were difficult to quantify. Therefore, the airflow quantity through the regulator can be expressed as:

\[ Q = C_c \sqrt{\frac{2 \Delta P}{\rho}} \frac{1}{\sqrt{1 - N^2}} A_r + Q_l \] (3)

where \( Q_l \) is the leakage quantity. Thus \( Q_l \) needs to be quantified. An approach to this modelling is developed.

Relationship Between Airflow Quantity and Regulator Resistance
The regulator can be treated as a set of two parallel airways namely:

1. The regulator opening and
2. The leakage paths, that is passages through and around the regulator other than the regulator orifice itself.
3. This can be illustrated as in Figure 4.

Therefore, the total resistance of regulator (\( R_t \)) can be modelled to consist of the regulator opening resistance (\( R_o \)) and the leakage path resistance (\( R_l \)). When the regulator is in a fully closed condition, the air flows through the leakage path only.

Airflow quantity through the regulator opening is calculated using the basic square law (\( \Delta P_x = R Q^2 \)). Based on this
equation and Equation 2, the relationship between \( R_o \) and \( A_r \) can be established as follows.

\[
\sqrt{\frac{\Delta P_s}{R_o}} = C_c \sqrt{\frac{2 \Delta P_s}{\rho}} \frac{1}{\sqrt{1-N^2}} A_r
\]

\[
\sqrt{\Delta P_s} \sqrt{\frac{1}{R_o}} = C_c \sqrt{\Delta P_s} \frac{1}{\sqrt{\rho}} \frac{1}{\sqrt{1-N^2}} A_r
\]

\[
\sqrt{R_o} = \frac{1}{C_c A_r} \sqrt{\frac{\rho(1-N^2)}{2}}
\]

\[
R_o = \frac{\rho(1-N^2)}{2C_c^2 A_r^2}, \quad \text{Since} \quad N = \frac{A_r}{A}, \quad \text{thus}
\]

\[
R_o = \frac{\rho}{2C_c^2 A_r^2} \frac{\rho}{2C_c^2 A^2}
\]

\[
R_o = \frac{\rho}{2C_c^2} \left( \frac{1}{A_r^2} - \frac{1}{A^2} \right)
\]

(4)

where \( A \) is the airway cross sectional area. Since this equation does not take leakage into account, the actual regulator resistance will be different from the one calculated by the equation above. Thus, actual resistance is \( R_t \). \( R_t \) is made up of \( R_o \) and \( R_l \) in parallel configuration and so the relationship between them can be established. Since \( R_o \) has been quantified by Equation 2, \( R_l \) has to be quantified also to allow \( R_t \) to be calculated. Thus, based on the measured pressure drop, the airflow quantity through the regulator can be determined.

To do this, \( R_o \) is first calculated using Equation 2, and then the total resistance is calculated using the square law based on the measured pressure drop and the measured airflow quantity. \( R_l \) then can be calculated using the parallel airways resistance relationship. Table 3 shows the calculated resistance of the regulator tested.

### Table 5 Regulator resistances

<table>
<thead>
<tr>
<th>Condition</th>
<th>( R_o ) Ns^2/m^8</th>
<th>( R_0 ) Ns^2/m^8</th>
<th>( R_l ) Ns^2/m^8</th>
<th>( A_r ) m^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully closed</td>
<td>38.65</td>
<td>( \infty )</td>
<td>38.65</td>
<td>0</td>
</tr>
<tr>
<td>1 board off</td>
<td>19.46</td>
<td>186.77</td>
<td>42.43</td>
<td>0.09</td>
</tr>
<tr>
<td>2 boards off</td>
<td>10.56</td>
<td>46.39</td>
<td>38.61</td>
<td>0.18</td>
</tr>
<tr>
<td>3 boards off</td>
<td>6.58</td>
<td>20.39</td>
<td>35.31</td>
<td>0.27</td>
</tr>
<tr>
<td>4 boards off</td>
<td>5.17</td>
<td>11.29</td>
<td>49.52</td>
<td>0.36</td>
</tr>
<tr>
<td>5 boards off</td>
<td>3.93</td>
<td>7.08</td>
<td>60.21</td>
<td>0.45</td>
</tr>
<tr>
<td>6 boards off</td>
<td>2.75</td>
<td>4.80</td>
<td>46.76</td>
<td>0.54</td>
</tr>
<tr>
<td>7 boards off</td>
<td>2.13</td>
<td>3.42</td>
<td>48.32</td>
<td>0.63</td>
</tr>
<tr>
<td>8 boards off</td>
<td>1.71</td>
<td>2.53</td>
<td>54.71</td>
<td>0.72</td>
</tr>
<tr>
<td>9 boards off</td>
<td>1.42</td>
<td>1.92</td>
<td>72.25</td>
<td>0.81</td>
</tr>
<tr>
<td>10 boards off</td>
<td>1.28</td>
<td>1.48</td>
<td>246.09</td>
<td>0.90</td>
</tr>
<tr>
<td>11 boards off</td>
<td>0.94</td>
<td>1.16</td>
<td>91.92</td>
<td>0.98</td>
</tr>
<tr>
<td>12 boards off</td>
<td>0.79</td>
<td>0.92</td>
<td>140.23</td>
<td>1.07</td>
</tr>
<tr>
<td>13 boards off</td>
<td>0.68</td>
<td>0.73</td>
<td>415.53</td>
<td>1.16</td>
</tr>
<tr>
<td>14 boards off</td>
<td>0.46</td>
<td>0.59</td>
<td>37.87</td>
<td>1.25</td>
</tr>
</tbody>
</table>

To quantify \( R \) a plot against regulator opening area was made, as shown in Figure 5. It was found that
\[ R_i = 32.734e^{1.1631A_r} \]. Therefore, the total regulator resistance, \( R_t \), can be calculated from:

\[
\frac{1}{\sqrt{R_t}} = \frac{1}{\sqrt{R_o}} + \frac{1}{\sqrt{R_i}}
\]

\[
R_o = \frac{\rho}{2C_v^2} \left( \frac{1}{A_i^2} - \frac{1}{A_r^2} \right)
\]

\[ R_i = 32.734e^{1.1631A_r} \]

And so the total regulator resistance, \( R_t \), can be calculated. The airflow quantity was then re-calculated using the square law based on the new \( R_t \). Results of this were then compared with measured values, \( Q_m \), as summarized in Table 4 and Figure 6.

![FIG. 16 - Quantification of resistance for leakage paths](image)

It can be seen from both the table and the graph that the difference is at all times less than 10 percent which is well within practical underground measurement tolerance and therefore this new equation is sufficiently reliable to be employed for further analysis.

<table>
<thead>
<tr>
<th>Condition</th>
<th>( Q_m ) m(^3)/s</th>
<th>New ( R_t ) Ns(^2)/m(^8)</th>
<th>New ( Q_m ) m(^3)/s</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully closed</td>
<td>2.05</td>
<td>32.73</td>
<td>2.23</td>
<td>-8.0</td>
</tr>
<tr>
<td>1 board off</td>
<td>2.53</td>
<td>17.49</td>
<td>2.67</td>
<td>-5.2</td>
</tr>
<tr>
<td>2 boards off</td>
<td>3.02</td>
<td>10.80</td>
<td>2.98</td>
<td>1.1</td>
</tr>
<tr>
<td>3 boards off</td>
<td>3.33</td>
<td>7.27</td>
<td>3.17</td>
<td>5.1</td>
</tr>
<tr>
<td>4 boards off</td>
<td>3.35</td>
<td>5.18</td>
<td>3.35</td>
<td>0.0</td>
</tr>
<tr>
<td>5 boards off</td>
<td>3.46</td>
<td>3.84</td>
<td>3.50</td>
<td>-1.1</td>
</tr>
<tr>
<td>6 boards off</td>
<td>3.62</td>
<td>2.93</td>
<td>3.51</td>
<td>3.1</td>
</tr>
<tr>
<td>7 boards off</td>
<td>3.75</td>
<td>2.28</td>
<td>3.63</td>
<td>3.4</td>
</tr>
<tr>
<td>8 boards off</td>
<td>3.82</td>
<td>1.81</td>
<td>3.72</td>
<td>2.7</td>
</tr>
<tr>
<td>10 boards off</td>
<td>3.86</td>
<td>1.17</td>
<td>4.03</td>
<td>-4.3</td>
</tr>
<tr>
<td>12 boards off</td>
<td>3.90</td>
<td>0.78</td>
<td>3.93</td>
<td>-0.8</td>
</tr>
<tr>
<td>14 boards off</td>
<td>3.89</td>
<td>0.52</td>
<td>3.68</td>
<td>5.7</td>
</tr>
</tbody>
</table>
FIG. 17 - Comparison between measured and new predicted quantity

From these the relationship between the regulator opening area and total resistance can be derived as shown in Figure 7. Based on this, pressure and airflow quantity relationships can be calculated from mine regulator impedance characteristic curves. These can be drawn for different mine configurations as shown in Figure 8. The three curves shown illustrate relationships from Table 4 for one, three and five boards off the regulator.

FIG. 18 - Relationship between new total resistance and regulator opening area.

An investigation was conducted to check whether the test method maintained accuracy with less measurement.

FIG. 19 - Drop board regulator characteristic curves.
data. It was found that with half the number of measurements taken (removing two boards at one time instead of one board) differences remained mostly less than 10 percent and the method was still considered reliable.

C-Section Regulator Tests

Similar tests were conducted on Drop Board style C-section regulators. Figure 9 shows a photographic view and the engineering drawing of a C-section regulator used by an Australian mine. The regulators were installed in either half or full sizes depending on the magnitude of the airflow regulation requirements and the locations.

FIG. 20 - Photographic view and engineering drawing of a half size C-section regulator

A half size ventilation regulator may consist of up to a total of 16 C-section galvanised steel boards which are secured with humpback split pins to the frame structure. The full sized version is two of these regulator frames placed side by side. The frame structure is secured in place using rock bolts to the concrete floor base. Packing is used to seal the drive around the regulator frame structure. Dimensions of each C-section board are 1.65 m in width and 0.2 m in height. The maximum opening area of a half size C-section regulator is 5.28 m².

Results of the initial test on one of the half size C-section regulators are shown in Table 5. Several test series were also undertaken to verify the relationship between the equivalent total regulator resistance, $R_t$, and equivalent opening areas, $A_r$ at various locations at the same mine. Figure 10 shows the calculated relationships between $R_t$ and $A_r$ for the C-section regulators. It can be seen that the relationships are similar to the relationship established from the drop board regulator tests.

Table 7 Results for one of the C-section regulator tests.

<table>
<thead>
<tr>
<th>Condition</th>
<th>$\Delta P_s$</th>
<th>$Q_m$ m$^3$/s</th>
<th>$Q_{cm}$ m$^3$/s</th>
<th>Difference</th>
<th>$R_t$ Ns$^2$/m$^8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully closed</td>
<td>385</td>
<td>6.3</td>
<td>0.0</td>
<td>n/a</td>
<td>9.70</td>
</tr>
<tr>
<td>1 board off</td>
<td>395</td>
<td>8.7</td>
<td>5.4</td>
<td>62.4</td>
<td>5.22</td>
</tr>
<tr>
<td>2 boards off</td>
<td>375</td>
<td>14.6</td>
<td>10.4</td>
<td>39.8</td>
<td>1.76</td>
</tr>
<tr>
<td>3 boards off</td>
<td>370</td>
<td>18.9</td>
<td>15.6</td>
<td>21.4</td>
<td>1.04</td>
</tr>
<tr>
<td>4 boards off</td>
<td>348</td>
<td>27.4</td>
<td>20.1</td>
<td>36.0</td>
<td>0.46</td>
</tr>
<tr>
<td>5 boards off</td>
<td>345</td>
<td>33.5</td>
<td>25.1</td>
<td>33.5</td>
<td>0.31</td>
</tr>
<tr>
<td>6 boards off</td>
<td>325</td>
<td>41.3</td>
<td>29.3</td>
<td>41.1</td>
<td>0.19</td>
</tr>
<tr>
<td>7 boards off</td>
<td>325</td>
<td>45.1</td>
<td>34.2</td>
<td>31.9</td>
<td>0.16</td>
</tr>
<tr>
<td>8 boards off</td>
<td>290</td>
<td>52.4</td>
<td>41.7</td>
<td>25.8</td>
<td>0.11</td>
</tr>
<tr>
<td>9 boards off</td>
<td>245</td>
<td>62.3</td>
<td>47.0</td>
<td>32.6</td>
<td>0.06</td>
</tr>
<tr>
<td>10 boards off</td>
<td>235</td>
<td>68.7</td>
<td>54.6</td>
<td>25.7</td>
<td>0.05</td>
</tr>
</tbody>
</table>
FIG. 21 - Relationship between $R_t$ and $A_r$ for C-section regulators.

In Figure 10, a theoretical relationship between the equivalent resistance and the opening area of a regulator similar to the approximation equation proposed by Le Roux (1990) was also included. The equation for calculating the size of regulator opening given the airflow quantity and pressure destroyed by the regulator is as follows.

$$A_r = C \times Q \sqrt{\frac{\rho}{\Delta P_S}}$$

where $C$ is a constant.

This equation is based on equation 2 with assumptions made to account for the general mining conditions, for example, $C = 0.64$ and $\sqrt{(1 - N^2)} \cong 1$. Therefore a value of 1.1 was suggested to replace these terms. As $R_t = \Delta P_s/Q^2$ rearranging the above equation, gives

$$A_r = 1.1 \times \sqrt{\frac{Q^2}{\Delta P_S}}$$

$$A_r = 1.1 \times \sqrt{\frac{\rho}{R_t}}$$

Therefore, $R_t$ can be calculated from the following equation at standard air density of 1.2 kg/m$^3$.

$$R_t = 1.452 A_r^{-2.0}$$

In Figure 10, it can be seen that the equivalent total resistance calculated from the tests was lower than the theoretical regulator resistance. As mentioned before, the equivalent total regulator resistance takes into account also resistances of the leakage paths which are parallel air paths through the regulator opening. It is expected that the values of $R_t$ will be lower than the regulator resistance itself.

Louvre Regulator Tests

Tests were carried out on louvre type regulators. This type of regulator is popular in Australian coal mines and an example is shown in Figure 11. Generally a double vehicle door design has louvres placed within panels of both doors. The louvre blades can be adjusted to various angles to control airflow. In this test both left and right doors could be set at nine positions. Tests were undertaken both by holding one side fixed and varying the other and by varying both sides. Table 6 gives a summary of one of these test results sets, with calculated resistance values and equivalent opening areas for the louvre settings. Relationships between resistance and the equivalent opening areas are shown in Figure 12.
FIG. 22 - An example of a louvre regulator.

Table 8 Example of the louvre regulator test results.

<table>
<thead>
<tr>
<th>Regulator Position</th>
<th>P_A (Pa)</th>
<th>Q (m³/s)</th>
<th>R (N/m²·m³)</th>
<th>A (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Angle</td>
<td>Right Angle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>9</td>
<td>2.8</td>
<td>76</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>8</td>
<td>14.5</td>
<td>86</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>6</td>
<td>31</td>
<td>115</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>5</td>
<td>45.2</td>
<td>172</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>4</td>
<td>55</td>
<td>200</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>3</td>
<td>65</td>
<td>235</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>2</td>
<td>76</td>
<td>298</td>
</tr>
<tr>
<td>1</td>
<td>89</td>
<td>1</td>
<td>89</td>
<td>387</td>
</tr>
<tr>
<td>1</td>
<td>76</td>
<td>2</td>
<td>76</td>
<td>225</td>
</tr>
<tr>
<td>1</td>
<td>65</td>
<td>3</td>
<td>65</td>
<td>125</td>
</tr>
<tr>
<td>1</td>
<td>55</td>
<td>4</td>
<td>55</td>
<td>83</td>
</tr>
<tr>
<td>1</td>
<td>45.2</td>
<td>5</td>
<td>45.2</td>
<td>50</td>
</tr>
<tr>
<td>1</td>
<td>31</td>
<td>6</td>
<td>31</td>
<td>32</td>
</tr>
<tr>
<td>1</td>
<td>2.8</td>
<td>9</td>
<td>2.8</td>
<td>20</td>
</tr>
</tbody>
</table>

It would appear that both aspects follow a similar relationship to that observed for the drop board or C-section regulators tested. There is no doubt that the resistance values increased as the equivalent opening areas decreased. However, it is suspected that due to the nature of fluid flow through louvre blade settings, the relationship between the resistance and equivalent opening areas would be more complex than drop board flow behaviour. A literature review on louvre regulators indicated that only limited research had been conducted on louvre flow behaviour.

FIG. 23 - Relationship between resistance and equivalent opening area for louvre regulator.
Roller Door Regulator Tests

Use of a roller door as a ventilation regulator was examined. Adjustment of the height of this door is undertaken via an automated control system. There is very little information on the use of roller doors as ventilation regulators and on the automation of such a system. Basically at the start of every shift the ventilation supervisor would remotely turn on face or booster fans and adjust regulator positions as required via the integrated mine monitoring and control system. Airflow and/or differential pressure sensors could be incorporated as part of such an automated control system to monitor changes made.

For this study, a roller door was installed for trial purposes as shown in Figure 13. Basically the roller door works like a Venetian blind, that is the lifting belt (made from sling material) is attached at the bottom corners of the door leaf and runs up inside the side guides to the drive assembly at the top. As the drive winds the belt, it lifts the bottom beam and each successive horizontal aluminium beam stacks on the top causing the vinyl coated fabric to billow out on the both sides as the air is expelled.

Several regulator characteristic tests were carried out to establish the relationship between \( R_t \) and \( A_r \). Figure 14 shows the relationship between \( R_t \) and \( A_r \) for the roller door tested. It should be noted that the air quantity and pressure was very low during the tests. Therefore, the calculation of equivalent resistance from measurements was difficult.

Also as one of the tests was conducted when the roller door was installed without a proper bulkhead built around it, a temporary brattice was erected to stop the airflows around the door. For this reason the leakage was substantial which means the equivalent resistance of the roller door regulator structure tested was much lower. As measurement was taken with the door fully closed it was possible to work out the equivalent resistance of the leakage paths around the door structure and then back calculate the resistance of the roller door only, \( R_{door} \), without the leakage paths as shown in Figure 14. It can be seen that it gives a much better comparison with the theoretical relationship between \( R_t \) and \( A_r \).

![FIG. 24 - Schematic and photographic views of a roller door regulator](image)

![FIG. 25 - \( R_t \) vs. \( A_r \) for Roller Door regulator tested](image)

Approx. Equation: \( R_t = 1.452A_r^{2.6} \), \( R^2 = 1 \)

- Door #1: \( R_t = 0.7066A_r^{-1.127} \), \( R^2 = 0.956 \)
- Door #2: \( R_t = 0.1265A_r^{-1.0724} \), \( R^2 = 0.927 \)
- \( R_{door} = 0.6195A_r^{-1.8016} \), \( R^2 = 0.9778 \)
It is proposed that further tests should be undertaken under higher air pressure and quantity conditions with proper bulkhead built around the roller door regulator.

CONCLUSIONS

Efforts to characterize or mathematically model a number of operating mine regulators have been described. Underground measurements have indicated that theoretical calculations to predict airflow quantity through practical mine regulators based on measured pressure drop are inadequate. The theoretical approaches are limited as they are based on prediction of fluid flow through a circular orifice in the middle of a plate whereas most mine regulators have a rectangular non-symmetrically positioned orifice. Also, most importantly, there is air leakage through the regulator bulkhead frame and gaps that increase actual quantity compared to that predicted.

The way to overcome this difference is to quantify the resistance of the leakage path based on regulator opening area and then recalculate the total resistance of the regulators. The relationship between leakage path resistance and regulator opening area varies, but the resistance should increase along with an increase in opening area. Based on measured pressure difference, the airflow quantity can be predicted accurately using the basic square law. It requires field measurement to quantify the leakage path resistance of each regulator, since each regulator has its own leakage characteristic which is affected by such things as size and number of gaps. This is tedious work, since the regulators can be set with many opening areas. However, it was found that with limited measurement data, prediction results are still accurate within acceptable tolerance appropriate to understanding mine airflows.

ACKNOWLEDGEMENT

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REFERENCES


UPDATE ON OUTBURSTS AND IN-SEAM DRILLING IN 2002

John Hanes ¹

ABSTRACT: There have been some developments in understanding the outburst mechanism and improving the control of in-seam drilling. Experience with techniques for draining from the surface has shown promise. There is a need for operators to provide facilities for more research so that the outburst phenomenon can be better understood.

INTRODUCTION

During 2002, there has been:
- Improved understanding of outburst mechanisms,
- Improved understanding of why the coal in some areas will not drain,
- A contribution of the usual hard-grind in-seam drilling for drainage and exploration,
- Some minor improvements in in-seam drilling technology from underground, but
- An increased effort in surface to in-seam drilling.

The information herein has been extracted, in some cases verbatim, from the notes of various workshops and seminars including the Coal 2002 pre-symposium Gas Workshop, Outburst Seminars and ACARP In-seam Drilling and Gas Workshops. Authors of papers are quoted, but the comments of many colleagues have been used with gratitude, but without specific reference.

MODELING

The aim of the ACARP funded research application of mathematical modeling by CSIRO (Choi, 2002 and Wold, 2002) is to try to get a better understanding of the mechanisms of outbursts using a simple mechanistic approach. In an outburst, as with pricking an inflated balloon, failure at the weak spot removes the energy barrier and allows the gas to expand, causing further failure of the membrane until the system reaches a new equilibrium (or stable) state. CSIRO have developed and used a coupled geomechanical-reservoir model to simulate outbursts. The model has shown that the coal deforms at a high strain rate after outburst initiation and as the coal continues to expand and disintegrate into smaller fragments, new surfaces are formed. Gas pressure around the new surfaces and in the voids, which are close and connected to the new surface, drop very quickly. The model has shown that the initiation of outbursts can be controlled by a number of factors. Much of the modeling to date has been based on data from Leichhardt and early West Cliff Collieries, ie 20 year old data. As the understanding of the mechanism improves, better data will be required for modeling and the willingness of collieries to help collect data will be essential.

Slater, and Yurakov, (2002) showed how mathematical modelling of gas reservoirs can provide practical assistance to facilitate gas emission evaluation and control during gate road development.

PERMEABILITY

Gurba, 2002 showed that, in the mines sampled for her ACARP project, the main difference between coal that does not drain and coal that drains easily is the nature of infilling of micro-cleats. Impermeable coal has micro-cleats which are typically infilled by carbonates and permeable coal has micro-cleats which are free from infilling. There is a need to understand the post depositional fluid flow and geology and precipitation of the carbonates. This appears to be a fertile area for research in an attempt to provide the link between microscopic and macroscopic features. The ideal outcome will be to use the microscope to help understand the causes of low

¹ ACARP
permeability so the mine geologist can better map areas of varying drainage potential in the mine. Micro-markers could also be developed as a useful exploration tool.

Robertson, (2002) stated “Permeability appears to be well understood in reservoir engineering terms, but it is still poorly understood in the coal industry and it is such a critical parameter. There are not enough permeability tests done. We let permeability get lost in the empirical approach we have to gas drainage”. If a reservoir simulator is used, or if it is necessary to design a gas drainage system, reliable information on permeability is required. It is then necessary to get real permeability testing including interference tests. An interference test requires at least three observation wells and a central pump well. On top of this is the cost of a pumping system and a monitoring system. Depending on depth and drilling costs, the total cost would be between $100,000 and $200,000. Permeability testing should be extended to the entire reservoir, including over- and under-lying seams and sandstone reservoirs, not just the working seam. In the Bulli seam, only around 10% of the gas comes from the mined seam. Stress measurement should be considered as part of a permeability test because the permeability is so closely controlled by stress. Relative permeability is very important.

The oil and gas industry has shown that the method of drilling can influence permeability tests. The hole skin factor can control desorption pressure. In-seam drilling is conducted mainly at or below gas desorption pressure. In such cases gas is desorbed uncontrollably while drilling and this can cause damage to the hole and can lead to bogging of the rods. If the hole is pressurized while drilling, the environment is quite benign. The Sigra borehole pressurization tool could be very useful here. In-seam drilling from the surface uses water pressure to stabilize the hole and drilling is easy compared to drilling under ground. Gas desorption only commences when the water pressure head in the hole is reduced below sorption pressure. According to Williams, (2002), when drilling from the surface into permeable coal, the weight of the drilling fluid can force fluid into the formation. Once the pore pressure is reduced to around gas desorption pressure, if the gas is migrating a long way to the hole and is carrying a lot of coal fines, a sudden drop in pressure can cause the coal pores and fractures to block and thus reduce the permeability. In permeability testing, it is necessary to reduce the in-hole fluid pressure very slowly to reduce hole damage. The problem of blockage is accentuated in friable strata.

Boucher, (2002) described the use of hydrofracture for in-seam holes at Dartbrook to increase flows from impermeable coal. Water fractures improved flows initially, but flows reduced in a few days to pre-frac levels. Sand fractures gave initial flows 20 times normal flow rates which appear to be sustainable over 6 months. Fractures were induced at 3 to 6 m spacings in holes.

HARD GRIND OPERATOR EXPERIENCE

Pryor, (2002) reported on the proposed upgrading of Tahmoor Colliery’s in-seam drilling capabilities for Tahmoor North. Previous drainage has been with holes 350 m long at 25 m spacings. For Tahmoor North, hole lengths will be 600 m. The longer holes require more powerful drills and the Mecca survey system developed by Longer exploration holes will also be necessary for testing seam structure. Dewatering of holes will also be required and a trial will be conducted of VLD’s tube feed roller system for introducing the dewatering tube into the holes. Tahmoor will change from multiple branches of holes to single holes to improved monitoring of drainage efficiency. Methods for maintaining borehole stability across dykes are being investigated.

Newman, (2002) reported that although borehole maintenance is a very basic need, it is often given a low priority. If maintenance is ignored, a lot of time and money can be spent forming boreholes which serve no purpose. Maintenance is required to fix problems of three main types - blockages by solid material, water removal and leakage. In many cases these problems can be avoided by applying good standards at the time holes are drilled and connected to the drainage system. The main aim of a drainage hole is to efficiently drain gas and to monitor the efficiency of a hole, it is necessary to monitor gas flows from the hole. Currently, gas flows are measured weekly in the early part of the hole, reducing to about once per month. Although automatic flow monitoring for the life of the hole would be ideal, it is not currently being conducted by any colliery. Under ACARP funding, Sigra developed a flowmeter with automatic monitoring, but no colliery has expressed sufficient interest to enable commercialization of the system.

Brown and Eade, (2002) reported that Tower Colliery has a 250 m wide structural zone which is nearly impossible to drill or drain. It is associated with a fault which varies between a thrust fault and a bedding plane fault. The coal in the zone contains 15 m³/tonne CH₄. The area is highly stressed with a prominent horizontal stress which has created enormous roof problems. Some intense bolting patterns with 8x8m fully grouted bolts per metre in the maingate roads is required. Attempts have been made to drill numerous drainage holes through the coal, but with little success. The zone is outburst prone and two outbursts occurred while remote mining was
conducted to cross it. The permeability of the coal is effectively zero in places. There is no gas flow from any holes which penetrate the zone.

SURVEY AND EXPERIMENTAL DRILLING

Verhoef, (2002) described recent advances in borehole surveying. These advances over the last 10 years occurred through close co-operation between collieries and AMT. Borehole surveying is approaching the ideal of a drilling guidance tool. There is still a need to incorporate some forms of “geophysical logging” into the drill guidance system so that geological changes in the coal seam can be detected and quantified during drilling. One of the major stumbling blocks in achieving this aim seems to be compliance. According to Verhoef, “The compliance issues severely impact on the technologies that can be applied. Aluminium cannot be used. Designs are restricted due to the total inductance and capacitance that can be used in combination with the battery voltages used. Current/Power restrictions i.e. resistors, zener diodes to prevent sparking. Enclosure strength because of flameproof requirement, add weight to designs and limits physical space available. It is very time consuming and difficult to obtain full compliance, in particular the State differences in paperwork, although both comply with Australian standards”.

Thomson, (2001) stated that current in-seam drilling technology is “sort of” providing a solution to outburst problems, but it is part of the problem. In-seam drilling for drainage is expensive, interferes with mining, provides insufficient lead times for drainage, has water and power issues. There have been no real advances in outburst detection methods and it is unlikely that detection will ever replace reduction of gas content. He suggested that medium radius surface to in-seam drilling, cheaper (less accurate) underground drilling and an analytical approach to in-seam drilling results should be considered as alternatives. With in-seam drilling, Thomson highlighted the need to monitor the drilling parameters for detection of structures and the development of in-hole geophysical tools. Any tools that reside behind the bit are prone to loss, especially when drilling in underbalanced pressure conditions. The risk could be reduced by drilling using water pressure in excess of desorption pressure. He expressed the opinion that the industry should consider a combination of oilfield rotary drilling technology and pump-down survey tools to reduce the cost of equipment at risk. Verhoef, in response, reported that AMT have developed the Drill Guidance System (DGS) which can incorporate natural gamma and other geophysical tools which might be developed in the future. They have also developed an IS computer for underground use. The expense of tools must be weighed against the benefits of the tool. A pump down tool takes time to collect data. A down hole tool used during drilling allows holes to be drilled more quickly and therefore more holes can be drilled.

A CMTE ACARP project C9020, Longhole Waterjet Drilling for Gas Drainage is due for completion at the end of 2002. The project combines pure waterjet drilling technology with conventional directional drilling technology. The final field trial is due to be conducted at Moura. CMTE are also involved in developing drilling technologies for soft and low permeability coals (ACARP project C10016). The system under development utilizes a combination of high pressure waterjet drilling and a casing advance system. For stimulation of impermeable coal, slotting equipment has been prepared. CMTE have made numerous attempts to secure a mine site for trials, but with no success.

SURFACE TO IN-SEAM DRILLING

Bos, (2002) described trials in surface to in-seam drilling conducted by Anglo Coal who experimented with tight radius drilling at German Creek and medium radius drilling at Moranbah North. “MRD will provide good exploration data with 9 holes sufficient to cover and hopefully predrain a 4 km block. If drilling were conducted up to 3 longwall blocks in advance, there should be sufficient time for drainage, good exploration data can be collected and near pure gas can be collected for sale. TRD could be as good as MRD if directional control while drilling can be obtained”. The value of surface to in-seam drilling could be greatly enhanced if geophysical logging of the in-seam section of the hole could be conducted.

Johnston, (2002) showed how surface to in-seam drilling complimented in-seam drilling to solve a longwall scheduling problem at Oaky North. Holes were drilled from the surface and turned in-seam to intersect vertical holes. The water lowering effect of these holes dramatically increased the flows in the in-seam holes drilled from underground resulting in no longwall delays.
GAS RESEARCH

Filipowski, (2002) described a novel and relatively inexpensive method of assessing face outburst proneness. “The gas composition of a coal sample changes with time. The proportion of each component relative to other components is transient. This is due to the different rates of desorption of the different component gases. Some gases desorb very quickly, eg the higher hydrocarbons and CO2. CH4 is slower and N2 is the slowest of the coal seam gases. Nitrogen remains in the coal for a long time. If you have a gas composition, you can assess the degree of coal degasification. This is a much more economical method of gas content assessment than desorption testing. It will not replace all content testing, but offers a quicker and less expensive method for infill testing in the mine”. He found that that if N2 is greater than 20%, the gas content will be below the outburst threshold. He hypothesised that Nitrogen can be used as a faster assessment of outburst potential than gas content.

Harvey, (2002) defined a major problem of outburst research “the outburst problem appears to have been solved. Outburst risk in the Bulli seam is deemed to be successfully managed through adherence to the threshold values”. It is difficult to examine outburst parameters if there are no outbursts. He emphasized that gas thresholds only relate to one aspect of outburst risk, gas content. Gas content thresholds, like any other standard, need to be analysed and reviewed on a regular basis and placed in the context of other contributing parameters. An understanding of the warning signs at the face is the fundamental final barrier. He commented “A number of us who are involved with outburst studies are concerned that the collective knowledge of the industry could be lost unless something is done to promote further research and document the knowledge. Without ongoing research and documentation, future generations of miners will have a steep learning curve.” There is also a need to make miners aware that drainage is not a panacea and that other factors such as warning signs at the face should be re-emphasised in training.

Eade, in a comment from the floor at the Coal 2002, pre symposium Gas Workshop, stated “Outbursts are seen to be under control, ie there is a fair factor of safety in gas content threshold values. In the factor of safety there is a cost component to productivity and safety… We need to continue research towards a fundamental understanding of outbursts. Until we have this understanding, it is difficult to go much further on a lot of the outburst parameters and put them into a threshold. The reason for outburst management success in the Bulli seam is the healthy safety factor with gas content.” If the gas content can be reduced to a manageable level, outburst should not occur from structure free coal. The structures are the focus for potential outbursts in otherwise drained coal. Techniques to reliably detect structures should be advanced.

A survey of collieries carrying out in-seam drilling and many individuals employed in the industry was conducted by the author as part of ACARP Project C10012 to assess the needs for ACARP funding of research into outbursts and gas drainage. No results are available at the time of writing.

CONCLUSIONS

There has been some progress in recent times in in-seam drilling and gas management, but in some cases industry has delayed taking up new developments or providing sites for research and has frustrated data gathering trials.

A great leap seems to have been made in overcoming initial inertia regarding surface to in-seam drilling. This technology will allow the use of non-IS equipment and bigger equipment to handle the task. Many operators express a desire to get drilling out of the pit. Drilling from the surface removes many barriers. There will be a change in thinking required to fund drainage from the surface several years in advance of mining.

There was not much good news from the researchers over the last year. Several projects have been frustrated by lack of colliery support for field trials.

Although the industry co-operates well to overcome problems associated with gas management, Tower Colliery clearly showed that Mother Nature can turn around and bite from time to time. In some cases it is necessary to walk away from problems.
REFERENCES

Bos, F., 2002; Surface to In-seam Drilling, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Boucher, C., 2002; Hydrofracture Experience at Dartbrook Colliery, in Gas and Coal Outburst Committee Seminar, Wollongong, 20th November, 2002


Choi, S.K., 2002; Understanding the Mechanisms of Outbursts Using a Mechanistic Approach, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Filipowski, A., 2002; The Role of Nitrogen in Gas Content Assessment, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Gurba, L., 2002; Gas Migration in Coal on the Microscopic Scale, in Gas and Coal Outburst Committee Seminar, Wollongong, 20th November, 2002

Harvey, C., 2002; Thresholds and Compliance Issues, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Johnston, M., 2002; Oaky North Surface to In-seam Experience, in ACARP Outbursts and Gas Drainage Workshop, Mackay, 25th October, 2002

Newman, R., 2002; Borehole Maintenance, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Pryor, G., 2002; Tahmoor North Outburst and Gas Management, in Gas and Coal Outburst Committee Seminar, Wollongong, 26th June, 2002

Robertson, B., 2002; Summation and Conclusions, Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Slater, M. and Yurakov, E., 2002; Coal Mine Development Gas Emission Modeling, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Thomson, S., 2001; In-seam Drilling Technology for Coal Seams Prone to Outbursts, in Gas and Coal Outburst Committee Seminar, Wollongong, 28th November, 2001

Verhoef, H., 2002; Surveying, in Control of Gas Emissions and Outbursts in Coal Mines, Coal 2002 pre-Colloquium Workshop

Wold, M., 2002; Outburst Research, Current CSIRO Directions, in ACARP Outbursts and Gas Drainage Workshop, Mackay, 25th October, 2002
SOUTHERN DISTRICTS EMERGENCY ESCAPE SYSTEM

Peter Baker

ABSTRACT: A new emergency escape system has been implemented into the coal industry in the Southern Districts of New South Wales. It has eventuated from a long process of trials and risk assessments and has involved every facet of the mining industry. A team of over 20 people had direct input into the development of this system, as well as the countless others who have helped refine initial ideas and avail themselves of a series of tests and trials. This paper The culmination of their combined efforts is presented.

INTRODUCTION

In September 1999, a new set of Regulations, Coal Mines (Underground) regulations 1999 (NSW Government 1999), under the Coal Mines Regulation Act, 1982 was introduced in New South Wales. Part of these new Regulations specified the escape equipment that must be supplied to all underground personnel. The Regulation specifies in Part 5 Clause 106 (1):

"A mine manager must provide sufficient escape equipment (including adequately maintained approved types of self rescuers) to allow safe egress of persons from the mine through conditions of reduced visibility and any irrespirable or irritant atmospheres that may be encountered."

McKenzie-Wood (200) completed an ACARP study on the availability, reliability and the use of Self Rescuers worldwide. The study included investigations in Australia, USA and South Africa, with a limited amount of information from Europe.

The results of the study led to two conclusions;

1. Inference that the new legislation leaves little room for the W65 Filter Self Rescuer in an escape system for potentially gassy mines, and

2. Although manufacturers are progressing through the issues, there is little to allay industry concerns over the reliability and durability of Self Contained Self Rescuers (SCSR) that rely on the chemical technology of Potassium Superoxide.

EMERGENCY ESCAPE

The new system now allows all mining companies to comply with recently introduced legislation. More importantly, it gives underground mine workers the best possible chance of detection, notification and escape in the event of a potentially devastating underground emergency such as fire or explosion.

In keeping with the NSW Department of Mineral Resources Guidelines MDG 1020 (2001), the system has a heavy focus on early warning systems such as real time gas monitoring and training of underground personnel in detecting change.

COMMUNICATION

Communication systems are vital at this point, and the use of Davis Audio Communication (DAC) and telephone communications to a central control room allows rapid dissemination of critical information to all underground personnel by use of a Personal Emergency Device (PED) system.

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1 Southern Mines Rescue Station NSW
CSE 100 SELF CONTAINED SELF RESCUER

All underground personnel wear a CSE 100 self contained self rescuer unit on their belt, which can be rapidly donned to protect against any irrespirable or irritant atmosphere. The unit has a rated duration of fifty minutes which will allow outbye personnel to readily reach a changeover or refill station. It also allows the use of self contained self rescuer escape right back to a place of safety if this is the employee’s preferred option. The unit was chosen because of its history in the US market (10 years without a major recall), its duration, the fact there is a twenty five minute unit with identical donning procedure, and its ‘starter’ mechanism which gives the wearer oxygen immediately upon donning (considered particularly necessary in outburst prone mining areas).

FIRST RESPONSE EMERGENCY EVACUATION KIT

The use of an air shower in the changeover process minimises the risk to personnel during changeover in a potentially irrespirable atmosphere. These will be located at the First Response Emergency Evacuation Kit (FREEK) as well as locations where SCSR to SCSR changeovers may be necessary (eg. L/W tailgate).

Spare SCSRs and enough Compressed Air Breathing Apparatus CABA suits for all personnel in the district are located at the FREEK, or first response station, which will be used by the crew as a meeting points. These are located at a designated point outbye the face (within a few hundred metres) and must be accessible from both intake and return roadways. The FREEK will also contain all equipment, such as brattice, pogo sticks, and stretchers necessary for response to an outburst at the working face.

COMPRESSED AIR BREATHING APPARATUS

At the FREEK, the self contained self rescuer is replaced by CABA, which will be worn by the underground personnel for the rest of their escape journey. A single cylinder, nine litre, three hundred bar Drager CABA suit is used in the Southern District. The single cylinder, 9L configuration was chosen because it is the same CABA configuration that has been used in the underground coal Mines which mine the Bulli Seam for over 10 years. The cylinders are Kevlar wrapped aluminium, capable of a filling pressure of 300 bar, giving an expected duration slightly greater than that of the SCSR.

The changeover technique needs to be well rehearsed, and requires the use of the positive pressure within the suit to clear any potentially fouled atmosphere from around a person’s breathing zone during changeover. Crews have undertaken initial training at the Southern Mines Rescue Station, followed by a ‘refresher’ training on site as the equipment has been installed. Further training is planned to be conducted ‘inseam’, as well as an annual refresher training at the SMRS.

After securing the CABA suit to the person and turning on the air supply, it is one last breath from the self contained self rescuer and the next from the CABA face mask. A manual ‘purge’ button on the suit allows the compressed air from the cylinder to be used to clear any fouled atmosphere around the user’s breathing zone. This of course means that at no time during the changeover are personnel exposed to potentially lethal atmosphere.

After securing the face mask, personnel move to the one quick fill outlet located at the FREEK to top up their suits before proceeding outbye. It therefore doesn’t matter how much air has been lost from the cylinder in the changeover process. The coupling from the outlet is securely connected to the quickfill attachment on the CABA suit, and using the beer tap arrangement, the pressure in the suit can rapidly be topped back up to three hundred bar. This will ensure all personnel have sufficient air supply to make their way outbye to a refill station with at least fifty bar left in their suit.

DISTANCE BETWEEN REFILLS

The distance between refill stations at Appin Colliery was determined by walk out trials and is currently one point two kilometres. The trials involved people attempting to escape in ‘worst case’ conditions (i.e. on foot in nil visibility but with Hefline as guidance system). The shortest distance traveled by any individual in these conditions before their warning whistle sounded (indicating 50 bar left in cylinder) was 1.5km. It was then decided to add another safety factor to the system and the 1.2km spacing was implemented.
ESCAPEWAYS AND GUIDANCE SYSTEM

The quickest and least demanding means of escape for the crew will be on transport, and this is still easily achievable wearing CABA suits. Transport also means the whole crew will remain together and their consumption rates (whether they are wearing CABA or SCSRs) will be substantially reduced.

If transport is not available and visibility allows, the travelling road is a good option as it is known to all personnel and usually provides better walking conditions. There is no lifeline however in the traveling road and reliance is placed on signs as indicators for refill stations and caches.

In cases of very poor visibility it may be necessary to escape down the return roadway using the tactile guidance system. A lifeline is located in each secondary escapeway consists of five millimetre diameter radio aerial which is held securely in place by rods attached to the roof or rib. The walkway beside the lifeline should be free of debris to allow a reasonable rate of travel without fear of slips, trips and falls.

Directional cones ensure the risk of total disorientation is minimised in cases of extreme visibility by allowing personnel to know with confidence they are heading to a place of safety. These cones are placed along the lifeline at nominal intervals of up to 100m. At places where there is an opportunity to move from the return roadway back to intake (i.e. trapdoors in overcasts or stoppings), there are 6 cones located back to back followed by a plastic disk. A ‘spur line’ then guides personnel to the trapdoor where an inspection is made followed by a decision on the most appropriate action.

REFILL STATIONS

The cascade refill system allows up to five people to refill their suits simultaneously without having to be exposed to the outside atmosphere. Timing of the refill varies between thirty seconds and three minutes, depending on the number of people attached to the refill unit. The refill units come in two different sizes though both have identical operating panels. The ‘C-20’ unit comprises 10 large ‘G’ size cylinders and has the capacity to refill 20 CABA sets from 50 bar (minimum pressure any person escaping would expect to arrive at the refill station with) back up to 300 bar. The ‘C-40’ unit comprises 20 large cylinders and refills 40 CABA sets. If there is a requirement for more than 40 refills at any one point (i.e. some outbye locations common to all panels escape routes), there would be more than one refill unit at the site.

The capacity at each of the refill stations provides the possibility of refuge for injured or fatigued crew members. Residual air in the large cylinders allows for many hours of refuge for up to 5 persons. Although the air supply remaining may be too low in pressure to continue to refill CABA suits to 300 bar, there is sufficient volume for 1 man to refuge for up to 40 hours (based on 35 litre/minute consumption rate) at the C-20 unit, and obviously longer at the C-40.

Telephone communication will be located at each refill station and should be used by escaping crews to obtain updates of the incident and inform surface control of current locations.

If the crew reaches a refill station and is not yet on the outbye side of the incident, escape continues to the next refill station, using the guidance system if required.

The system will continue to guide the crew to a designated place of safety, which is anywhere on the outbye side of the problem, or to a place that needs to be determined at each mine that has been determined as a place of guaranteed fresh air.

ADVANTAGES

The major advantages of the escape system are:

- The PED communication system allowing quick dissemination of critical information to all underground personnel
- The guidance system which the trials proved was necessary to guide experienced personnel through underground workings that were familiar to them before poor visibility was encountered – disorientation can be equally as deadly as a fouled atmosphere.
CABA is preferred by the workforce because of the cool air supplied by the set and the minimal breathing resistance.

CABA also allows communication between the crew as they are escaping, as well as with surface control.

Crews can train with actual equipment, rather than rely on simulation of the system as has been used in the industry with Filter Self Rescuers and Self Contained Self Rescuers.

The system is simple to use giving appropriately trained personnel the ability to operate in high stress circumstances.

The system allows both a refuge and response option, and could possibly be linked into first response or rescue teams.

The system now gives the underground workforce in the Southern Districts the best possible chance to escape to a place of safety in the event of an emergency which adversely alters the mine atmosphere.

CONCLUSION

The introduction of the Southern Districts Emergency Escape System is one of the largest changes to the underground mining industry in recent times – akin to the introduction of the Filter Self Rescuer in 1966. The introduction process will not be without problems and these will be dealt with as they arise. It is important though, that right through this process sight is not lost of the reason such an elaborate escape system is being introduced – it is for the same reason our industry has made many changes throughout the years – to make the underground mining industry a safer place for all who sail in her.

REFERENCES

Mc Kenzies-Wood, P, 2000, the performance and selection of self rescuers, ACARP Project No C10002.

NSW GOVT, 1999 Coal Mines (underground) Regulations (NSW GOV Printer: Sydney).

CONTROLLING AND REDUCING HEAT ON LONGWALL FACES

Philip Mitchell

ABSTRACT: In recent years uncomfortably high ventilation temperatures have become more common on longwall faces in Australian coal mines. Increasing strata temperatures at relatively shallow depths in combination with high surface ambient temperatures, particularly in Queensland, have led to high intake temperatures. These have approached trigger levels that introduce reduced face operator exposure times. With the addition of heat from coal breakage and goaf caving on high production longwall faces the working environment has become uncomfortable and continuous exposure over a shift is potentially injurious to health. Typical strata temperatures at 200m depth are 35°C increasing to 38°C at 350m depth. The added heat from broken coal and rock on the face and in the goaf together with heat from machinery, that has been progressively increasing in capacity, results in wet bulb temperatures exceeding 30°C and humidity of 95% to 100% on longwall faces. Management plans have introduced controls to limit continuous working times for personnel on longwall faces in the hot and humid conditions. This impacts on productivity and in some situations requires additional personnel in the panel crews.

Increased ventilation quantities are a partial solution because evaporative cooling rates and reduction in effective temperatures are minimal in high humidity conditions and less effective in the already high air velocity currents on faces. High air velocities also introduce other face environment problems with dust, increased pressure differentials and goaf leakage quantities which re-enter as additional warm air back onto the face. Spontaneous combustion risks also increase in thicker seam environments.

Depending on seam conditions more attractive approaches can be used such as three heading longwall development allowing a back return airway using the goaf as a partial heat sink and the introduction of direct cooling of air in the longwall panels by spray systems without disrupting the passage of employees and equipment.

INTRODUCTION

Hot and humid conditions in Australian coal mines have historically been associated with poor ventilation practices in face zones even though legislation has prescribed minimum requirements for air velocities and upper limits for the effective temperature. The introduction of higher capacity longwall equipment, longer panels and wider faces as well as alternative shift rosters over the last ten years has required a revision of the standards for the management of the underground environment.

The prevention of heat stress has become a major focus for longwall operations, particularly in the hotter climate of Queensland. This has led to the introduction of the Approved Standard for Management of Heat in Underground Coal Mines (QMD 99 7460) in 1999 following events demonstrating the unworkability of the existing legislation under extended shift working hours and the advances in heat stress knowledge and management.

The management of hot and humid conditions is principally achieved by increasing air quantities on the longwall face to reduce the effective temperature and introducing periodic rest periods and rotating work duties. Increasing the air velocity introduces problems with dust, pressure differentials across the face and goaf edge and less effect on reducing the wet bulb temperature in already high air currents. The benefits of increasing air velocities above 2 m/s diminish in hot environments (D Mitchell 1999).

An understanding of the limitations of the common heat stress indices as guides to safe working limits for workmen on high production faces and the principal heat sources is necessary when developing methods for controlling the work environment.

1 Minarco Pty Ltd, Sydney
Alternative longwall ventilation patterns to assist in removing heat generated at the face and spot cooling of intake air are practical and effective methods to reduce hot conditions. To implement these systems the application of three heading panels rather than the traditional two heading development is a recommended approach for modern high production panels over 3 km in length.

**THRESHOLD LIMITS AND HEAT INDICES**

In discussing methods of controlling and reducing heat input into longwall ventilation it is necessary that a benchmark be established for designing the ventilation system. A common upper temperature limit that has been used for design limits is an effective temperature of 28°C (Pickering, Tuck 1996; Graveling, Morris and Graves, 1988). The effective temperature is the most common heat index used in the underground coal mines. The maximum allowable effective temperature is 29.4°C with reduced work times generally imposed by Heat Management Plans between 27.2°C and 29.4°C. The management schemes in place impose regular rest periods when the temperature is between these two limits.

However heat stress depends on a number of factors other than the effective temperature, notably the work rate, clothing worn and acclimatization of the person. The wearing of personal protective equipment is a barrier to body evaporative cooling and is not accounted for in the assessment of the effective temperature indices. Additionally the effective temperature scale is not suitable for high velocity areas. The effective temperature scale does not take into consideration radiant heat. Although radiant heat in mines is generally considered minimal compared to convective heat sources (Schneider, 1999) there is a definite radiant heat flux from the goaf that is experienced by face operators when at the rear of supports in the low air velocity regions. This may explain the higher mean metabolic rate recorded for a fitter working on a longwall face (Tranter, 1998).

A more common index used in industry and recognised by the ISO and national health authorities is the Wet Bulb Global Temperature (WBGT). This index includes the wet bulb temperature and the radiant heat within the work environment. The relationship is

\[ \text{WBGT} = 0.7 \, t_{wb} + 0.3 \, t_g \]

Where

- \( t_{wb} \) = wet bulb temperature (°C)
- \( t_g \) = globe temperature (°C)

Tables have been prepared that equate the WBGT to work load and the recommended rest periods. It is a universally accepted method and can be readily applied to work procedures at longwall faces. Table 1 shows the recommended Threshold Limit Values for standard work load classifications. The work load expressed in W/m² is the metabolic heat produced by the body (Watts) when undertaking a physical activity expressed in terms of the body surface area. The typical body surface area is taken as 1.83m².

<table>
<thead>
<tr>
<th>Work Load (W/m²)</th>
<th>Work Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Continuous</td>
</tr>
<tr>
<td>Light &lt; 120</td>
<td>30.0</td>
</tr>
<tr>
<td>Moderate 120 - 190</td>
<td>26.7</td>
</tr>
<tr>
<td>Heavy &gt; 190</td>
<td>25.0</td>
</tr>
</tbody>
</table>

OSHA Technical Manual, Section III, Chapter 4.

The classification ranges for work loads varies and references for hard or heavy work provides metabolic heat loads from 175 W/m² to above 345 W/m² (Commonwealth Department of Health 1980, Pickering et al 1996). The WBGT is heavily weighted by the natural wet bulb temperature and in high humidity conditions or where evaporation of sweat is restricted the reliability of the WBGT index is limited.

Therefore both the most common heat indices used in coal mines have their limitations (Bethea and Parsons, 2002). For ease of measurement on longwall faces the effective temperature determinations continue to be used in mines and values of Basic Effective Temperature (BET) can be substituted for WBGT. Reference values for both indices are representative of the mean heat effect over a long period of work and it is necessary for judgment to be made for work exposure times during heavy physical work.
Applying the correct rest breaks for the longwall face operators depends on the knowledge of the work activities and metabolic heat generation. Mines that adopt effective temperature indices use nominal temperatures for determination of rest periods without specific reference to the work activity. Insufficient studies have been conducted to determine the heat transfer rates for the various work classifications on modern high production longwall faces in Australia. One study carried out at mine in central Queensland (Tranter, 1998) provided the results shown in Table 2.

Table 2 Metabolic Work Rates for Longwall Operators

<table>
<thead>
<tr>
<th>Classification</th>
<th>Metabolic Work Rate W/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel fitter</td>
<td>131</td>
</tr>
<tr>
<td>Support operator</td>
<td>70</td>
</tr>
<tr>
<td>Shearer driver #1</td>
<td>102</td>
</tr>
<tr>
<td>Shearer driver #2</td>
<td>120</td>
</tr>
</tbody>
</table>

Peak metabolic rates at up to 300W/m² were recorded (Abt and Tranter, 1999) in the tests. It was also stated that little face production was carried out over the period of the tests. More studies are required to obtain realistic ranges for face operators, however it can be stated that for wet bulb temperatures above 26°C periodic breaks may be necessary.

There is no direct scale equating WBGT to effective temperature, however in their literature review, Graveling et al equated a WBGT of 28.2°C to 26.8°C BET. From Table 1 and Table 2 this shows that for typical work rates on a longwall face the maximum BET for continuous work is closer to 26°C. However the BET is developed for essentially nude men. Therefore an allowance for clothing must also be considered.

Whichever heat index is selected it is considered that a more conservative initial trigger value for job rotation or rest periods be adopted due to the combination of clothing effects and the uncertainty of metabolic work rates, particularly with the variable conditions along the face.

In the months from October to March the mines in central Queensland experience high surface temperatures and combined with increasing strata temperatures and additional sources of heat in the underground workings. The nominal wet bulb temperature of 26°C is often exceeded.

**SOURCES OF HEAT IN LONGWALL OPERATIONS**

**Typical Surface Conditions**

The temperature on longwall faces is influenced by numerous sources both in the vicinity of the working area and by conditions external to the face. In the longwall mines operating in the central Queensland Bowen Basin area, face conditions that have been tolerable during the cooler months of the year become hot and humid conditions in summer. The additional heat source is the high surface and intake air temperatures entering the mine. Dry bulb temperatures exceeding 35°C are experienced for more than forty days during the months from November to March. Figure 1 shows the twelve month mean dry and wet bulb temperatures in central Queensland (Bureau of Meteorology Australia). The comparative low wet bulb temperatures provide a measure of comfort as it is the wet bulb temperature which has the major influence, along with air velocity, on the cooling power and comfort of the working conditions. However for approximately six months of the year the wet bulb temperature is often between 20°C to 23°C which will allow only a rise of as little as 5°C before rest breaks are required. As the intake air flows through the roadways there is a complex interplay of moisture increase through evaporation with an associated vapour pressure increase influencing the wet bulb temperature. There is a reduction in the dry bulb temperature and then a steady increase as strata temperatures and depth increases. A typical trend of measurements taken in mines is shown in Figure 2.
With increasing lengths of longwall panels the intake air is approaching 26°C to 27°C wet bulb at the longwall stage loader. There is very little margin available before heat management action must be implemented on the longwall face.

**Temperature Increases Underground**

The sources of heat in underground coal mines are well documented (Pickering and Tuck, 1996; Whittaker, 1979) however the following are noted for typical modern Australian longwall operations.

- The high rate of increase in strata temperatures at relatively shallow depths for the Bowen Basin coal mines;
- Autocompression which increases the wet bulb temperature by approximately 0.4°C per 100m, depending on the surface wet bulb temperature;
• Rapid production and therefore release of heat from broken coal and rock at the working face and within the goaf immediately behind the face;
• High face air quantities and pressures with consequently larger volumes of air sweeping the goaf behind the face and returning onto the face at various locations along the face at near to the strata temperature;
• Increasing equipment power with accompanying heat dissipation;
• Two heading development using a single intake traveling road and homotropal belt roadway and single return for longwall operations;
• Down dip advance of workings with consequent auto-compression and strata temperature increase.

The most significant of these sources of underground heat are discussed below.

**Strata Temperatures**

The heat gained or lost by the ventilation at the roadway perimeter is determined by the difference between the air temperature and wall surface temperature. The surface temperature is difficult to estimate and relationships have been developed between virgin strata temperatures, age of roadway, size of opening and the thermal properties of the surrounding coal and rock. The wetness of the airway walls influences the rate of latent heat evaporation and rise in wet bulb temperatures. Charts are available for determining the various coefficients to assist in calculating the surface temperature of airways. Along the older airways in main intakes the strata surface temperature is less than the summer air temperature and sensible heat is transferred to the surface reducing the dry bulb temperature. The most recently excavated airways in longwall panels will reverse the sensible heat transfer back into the air stream with the dry bulb temperature rising. The wet bulb temperature, which is the most important in assessing the ability for the body to cool, increases at 1.0 to 1.5 °C per km depending on the age of the panel development which may vary from months to two years over the period of longwall operation. It also depends on the degree of wetness and the presence of flowing water. For panel lengths now exceeding 3 km the wet bulb temperature increase is as much as 4.5°C along the length of the panel.

The rate of heat flow into the ventilation after a specific time following rock exposure per unit area of exposed rock is given by:

\[ q/a = v \left( \frac{kwC}{\pi \theta} \right) (t_{vr} - t_{db}) \]

where
- \( q = \) heat flow energy (W)
- \( a = \) area of exposed rock (m²)
- \( k = \) thermal conductivity of strata (W/m °C)
- \( w = \) density of rock (kg/m³)
- \( C = \) thermal capacity of strata (J/kg °C)
- \( \theta = \) time since rock exposed (seconds)
- \( t_{vr} = \) virgin strata temperature (°C)
- \( t_{db} = \) dry bulb temperature of air

Larger air quantities will reduce the rate of increase in the air temperature, however unless the mine ventilation system and fan selection has been made in anticipation of high air quantities and associated pressures, particularly for single intake and return layouts, there is a limitation on the ability to increase the air flow into a longwall section. Other factors will also impose limitations on longwall face quantities such as dust and maintaining pressure differentials across the goaf within acceptable limits.

**Rapid Production**

Longwall mines have increased production rates over the last two to three years to beyond 5 Mtpa to 6 Mtpa. Over the last ten years the rate of production, taking into account face availability, has increased from approximately 700tph to more than 1400tph with peaks up to 3000tph. For strata temperatures of 35°C the heat liberated by the broken coal at a typical rate of 350kg/s can be calculated from:

\[ Q = M \times C \times (t_1 - t_2) \]

Where
- \( Q = \) heat flow (W);
- \( M = \) mass flow of coal (kg/s);
C = specific heat of coal (J/kg °C);

\( t_1 \) = temperature of broken coal after cutting (°C);

\( t_2 \) = temperature of broken coal along panel intake (°C).

The temperature \( t_2 \) will be influenced by the air velocity, wet bulb temperature, traveling speed of the coal conveying system, wetness of the coal, the relative velocity between air and coal and the fragmentation of the coal. The use of motor cooling water for sprays along the stage loader and crusher contribute to increased wet bulb temperatures with the wet and dry bulbs depression typically less than 1°C at the last cut through of the maingate conveyor.

For coal with a typical specific heat value of 850 J/kg °C and assuming a 50% reduction in virgin coal temperature along the length of the maingate conveyor and a strata temperature of 35°C, the heat load into the ventilation is approximately 1200 kW. From the above equation the air temperature would rise by approximately 7°C. This is not the situation in practice where temperature rises along maingate intake airways have been recorded at least 2 to 3°C.

It is therefore now common practice to use a homotropical ventilation system for the maingate roadway. The single intake traveling road in the longwall panel is therefore required to ventilate both the longwall face and the maingate roadway. This typically requires 50 to 70m³/s of air depending on the face dimensions and panel length.

**Heat from Goaf Material and Oxidation**

The collapsing goaf associated with rapid extraction rates presents a greater source of heat than the cut coal on the face. The rock or coal surface area is rapidly increased and the surface is considered to be at virgin strata temperature. Leakage airflow rates reach an equilibrium temperature similar to that of the virgin rock temperature. Typical goaf leakage quantities have been found to be approximately 20% of total face volumes and this has been indicated by studies (Longson and Tuck, 1985). For a face quantity of 40m³/s as much as 8m³/s would flow behind the supports re-entering the face at various locations but mostly near the tailgate. This air has been measured when emerging at the tailgate totally saturated at 33°C where the strata temperature is 34°C.

Oxidation of coal in the goaf produces heat which will add to the strata heated air leaking through the goaf. For each 1kg of oxygen consumed 12,675kJ of heat is produced. Therefore typical air quality measurements in returns showing an oxygen depletion of 0.2 to 0.3% would add as much as 400kW of heat. Much of this heat is retained in the goaf and is partially removed by the air. However, increased face quantities producing larger pressure differences and therefore more leakage, particularly further into the goaf, will carry additional heat back towards the tailgate end of the face.

**Heat from Machinery**

The increase in the nameplate power rating of the longwall equipment has accelerated over the last twenty years. Previous heat studies of longwall faces (Whitaker, Fiala et al) studied longwalls with face production rates less than 2000tpd and longwall equipment powers in the order of 600kW. The majority of the energy consumed by the electrical machinery is dissipated as heat. Thermodynamically, the only work is that against gravity. Total face power on modern longwalls is now above 4000kW and, assuming an overall operating rate at 60% of total nameplate power, with as much as 70% of the energy converted to heat an approximate heat load is 1680kW. Most of this heat is conducted away with motor cooling water however unless suitably disposed from the face it will eventually add to the temperature of the air stream. The dissipation of this heat is not instantaneous but is becoming a major source of heat at the working face.

The effects of autocompression, strata temperatures and strata water flow into the roadways plus stationary machinery such as conveyor drives, for longwall panels over 3km in length and at 250m depth of cover the wet bulb temperature can increase from a typical summer surface value of 22.5°C to 27°C at the intake side of the longwall face. The impact of machinery, goaf strata and oxidation can add another 5°C to the wet bulb temperature along the face.

In recognition of the limits placed on working times by the adopted standards and the inevitable progress towards deeper workings and higher capacity mining equipment it is apparent that alternative methods of ventilation have become necessary.
AN ALTERNATIVE LONGWALL VENTILATION ARRANGEMENT

Development

Two heading longwall panel development has been the established norm for many decades. The subsequent longwall ventilation is invariably by a single return and twin intakes or a single intake with homotropal belt road ventilation. This is sufficient for longwall ventilation as a classic U system. Gassier mines would establish a bleed return from the intake side around the goaf. However, without a return system around the goaf, the ventilation of the single section of roadway that remains as the longwall retreats requires boreholes or fans. There have been instances where the ventilation of this section of roadway has been by leakage back through the goaf to the longwall return.

A three heading longwall panel development provides many advantages for introducing additional ventilation options as well as operational advantages for the location of service equipment.

The development of two heading panels beyond 3.5km requires high ventilating pressures and when this method is combined with a simple U system of face ventilation and homotropal maingate ventilation a 7km single roadway results with ventilation pressure demands exceeding 900Pa.

A three heading longwall arrangement allows:

- The longwall maingate conveyor to be set up as a homotropal return retaining two intake airways into the panel. This provides increased ventilation capacity above that of a two heading section for less ventilating pressure;
- The second intake roadway can be used as a “heat sink” roadway when refrigeration is considered to be a necessary option above that of air velocity for combating hot conditions; or
- A direct contact cooling water spray station can be established in the one principal intake roadway while the second intake allows bypass for vehicular traffic around the fixed water spray station.
- Two longwall return airways with the ability to establish a back return roadway from behind the longwall face. This will provide a separate path for the face leakage behind the supports and not have this hot and humid air coming back onto the face, particularly in the walkway behind the support legs;

A suggested layout for the ventilation arrangement is shown in Figure 3.

The combined U system with a secondary back bleed return allows heat released from falling and broken goaf material and the exothermic oxidation of coal within the goaf to be directed via the face leakage flows away from the face. The volume and nature of the flow interaction between the face and goaf leakage depends to some extent on goaf compaction, however it has been found from numerous observations that leakage tends not to re-enter the face area until close to the tailgate roadway. A significant heat effect on workmen along the face is the radiant heat coming from the goaf material which can be felt in between the supports. Results referred to in Table 2 for a fitter demonstrate the potential threat from a combination of radiant heat and the low air velocity region towards the rear and base of supports when carrying out maintenance on supports. For a face length of 250m and an estimated strata collapse zone up to 10m the heat released into the immediate 20m of goaf is quite significant. This is more than sufficient to maintain a goaf atmospheric temperature at the virgin strata temperature and raise the wet bulb temperature to that approaching the strata temperature. These effects can be readily measured at the goaf edge near the tailgate supports. By directing the flow of air alongside the goaf to the next cut through will prevent this hot and humid air entering the face towards the tail end and into the return roadway itself where in many instances persons are required to work setting secondary supports.
In thick seam operations the potential risk of spontaneous combustion needs consideration. Numerous articles have been written on the advantages and disadvantages of bleed or back return systems. Advantages in controlling methane in spontaneous combustion sensitive conditions using back return systems as opposed to bleed airways has been shown to be effective (Highton, 1979, McKensey and Rennie, 1988). More recent developments in goaf sealing technology and atmospheric monitoring enables this method of longwall ventilation to be seriously reconsidered to control heat convection from the goaf. The risk has been diminished by:
1. Increased rates of longwall extraction reducing the duration that air flows over sections of the goaf. Longwall retreat rates of 10m per day allow cut throughs alongside goaves to be progressively sealed within two weeks limiting the potential oxidation period.

2. Goaf seal technology has advanced significantly over the last ten years with the development of monolithic structures that are constructed to specific design criteria. The leakage through goaves due to poor sealing has been practically eliminated by these structures. The air movement through the goaf is influenced more by the ventilation pressure difference across the face which has been increasing in a desire to limit the effective temperatures. Well constructed seals have resistance values in excess of 50,000 Ns$^2$m$^{-8}$.

3. Continuous monitoring of the atmosphere behind seals and in longwall return airways together with regular gas sampling and analysis has increased the knowledge of and trends in goaf atmospheric conditions.

Progressively sealing the back return airway will maintain a positive pressure differential between the face and the open cut through.

**Refrigeration of Intake Air**

Refrigeration of intake air is currently being trialed at mines in the Queensland during the hotter months of the year when the intake temperature often exceeds 35°C dry bulb. The plants are not permanently installed and do not chill the total mine intake capacity.

Cooling the mine ventilation can be achieved by direct cooling of the air by chilled water sprays, indirect by cooling coils or a combination of both. Deeper coal mines have employed both methods (Hamm E, 1979). The economics of underground versus surface cooling plants depends on a number of factors however the most significant are depth and the distribution of the workings. A generalization by Ramsden and Carvahlo (1988) for gold mines was that at depths to 2000m there was no clear advantage of either system. There are however many advantages of installing a cooling plant on the surface not least of which is the dissipation of heat from the heat exchanger which would necessarily be in a return airway underground.

For the relative shallow depths of Australia’s longwall mines and concentration of workings a direct cooling system using reticulated chilled water is ideal. A lower capital cost for piping, less pumping costs and an easily expanded system with mining make this method of cooling economically attractive.

**Direct Cooling Systems**

Over recent years the trend for the cooling of the mine climate at the larger metalliferous mines is to install bulk air refrigeration plants on the surface. This provides an advantage for plant maintenance, larger installed capacities, heat dissipation and, if required, the circulation of chilled water. In consideration of coal mines and longwall faces, bulk air cooling plants for the mine intake air is inefficient with up to 30% of the mine ventilation being lost through leakage. Alternatively piping of chilled water underground to the working areas enables the air to be cooled for maximum effect near the longwall face. Insulation of these pipes is sometimes required to avoid water temperature rises although exposed pipes does have some benefit in cooling the intake air stream.

The chilled water can be delivered anywhere in the mine and can either directly or indirectly be applied to cool the intake ventilation. An indirect method of chilled water cooling using coils allows the water to be more easily managed and re-directed to other areas without entering onto the traveling road. However, cooling efficiency is compromised requiring higher water flows and the periodic cleaning of the coils of dust.

Direct air to water contact using spray chambers provides a more efficient cooling method. Applying a spray system closer to the longwall working area will require less water and power at the refrigeration plant. The advantage of a three heading longwall panel provides a second intake airway where a series of counterflow spray chambers can be installed with appropriate water sumps and pumping equipment. The majority of the intake air can pass through the chamber by erecting vehicle doors in the parallel intake. An arrangement is shown in Figure 4. For a total longwall panel intake quantity of 60m$^3$/s of which 50m$^3$/s is directed through the spray chamber at an intake temperature of 27°C dry bulb and 26°C wet bulb a reduction to 21°C Dry bulb/Wet bulb can be readily achieved with 20 L/s of chilled water entering the chamber at 10°C. The estimated volume flow of chilled water to reduce the heat capacity in the air can be obtained by using the following energy equations balancing the water flow heat gain and the change in :sigma: heat of the air current.
\[ Q_w = M_w \times C_w \times (t_{w2} - t_{w1}) \]
\[ Q_a = M_a \times (S_1 - S_2) \]
\[ \eta = \Delta t_w / (t_{wbi} - t_{wi}) \]

Where
\( Q_w \) and \( Q_a \) = thermal energy of air and water respectively (kW)
\( M_a \) and \( M_w \) = mass flow of air and water respectively (kg/s)
\( C_w \) = specific heat of water (kJ/kg °C)
\( t_{w1} \) and \( t_{w2} \) = temperature of chilled and outgoing water respectively (°C)
\( S_1, S_2 \) = sigma heat of air before and after the chamber (kJ/kg)
\( T_{wbi} \) = the wet bulb temperature of the incoming air (°C)

The change of thermal energy in the air is 1000 kW.

For a water efficiency (\( \eta \)) of 0.65 and an inlet water temperature of 10°C the outlet water temperature would be approximately 20.4°C. The ventilation pressure loss across a spray chamber will depend on the baffle or eliminator plate configuration to remove water droplets picked up by the air. For ventilation estimations approximately 250 Pa should be allowed. With this pressure loss the total fan pressure consumption around the longwall section is still below that of a twin heading development longwall of 3km length for similar face quantities.

Using a typical maximum rate of increase of 1.0°C per km for the wet bulb temperature the estimated wet bulb temperature at the last cut-through before the longwall face would be approximately 24°C. In longwall panels that are 4.5km to 5km in length the age of the initial 2km of intake roadways would be at least 6 months and have a low impact on the air temperature increase as long as they were kept reasonably dry. Wet bulb temperatures at the intake side of the longwall face are therefore estimated to not exceed 24°C.

The water collected at the spray chamber is pumped into the supply line to the face for dust suppression and motor cooling water. This provides additional cooling capacity at the face. Water consumption for dust suppression and motor cooling on high capacity faces is typically 10 to 15L/s, the majority being used by the shearer with total rated power up to 1500kW. Excess water can be either directed to other underground operations or pumped back to the surface for re-use. Float switches and PLC systems control the pumps and valves depending on demand. A schematic of the water circuit is shown in Figure 4.

Due to typical underground climates the mine cooling system will be required to operate less than five months of the year and water demand for the direct contact cooling sprays can be varied depending on daytime temperatures.
CONCLUSION

Longer panels and hot surface climates have combined with the traditional sources of mine heat to produce uncomfortable working environments on high production longwall faces. Increasing panel air quantities and face velocities has provided respite from these conditions, however further benefits are now limited due to the already high velocities and associated pressure differentials across the face and goaf. Alternate ventilation arrangements using three heading panels to provide face and panel ventilation options with less pressures and introducing reticulated chilled water cooling systems offer a way to maintain the trend of increasing longwall productivity without discomfort to the operators.
REFERENCES


APPLICATION OF BULK COAL SELF-HEATING TESTS TO LONGWALL OPERATIONS

Basil Beamish ¹, John Phillips ², Mick Brown ¹ and Danny Millers ¹

ABSTRACT: A new laboratory has been established in the Division of Mining and Minerals Process Engineering, which has refurbished and recommissioned a 2-metre self-heating column built at The University of Queensland (UQ). This equipment overcomes the limitations associated with previous large-scale testing. Repeatable test results are achievable within days instead of months and are far more advanced than any previous work at this scale. The column is ideal for simulating goaf behaviour and for teaching the fundamentals of heating development, including gas detection and analysis. Seventeen test runs have now been completed since the initial recommissioning test in late 2001, with a 100% success rate. Results to date clearly show that moisture transfer is the key factor in coal self-heating development. There is a critical moisture content below which coal oxidation and resultant self-heating is inevitable. This has significant implications for detecting and reducing the risk of a heating in longwall operations. Dried coal as a result of gas drainage will be more susceptible to self-heating as it is predisposed to both heat of wetting from moisture adsorption and accelerated coal oxidation due to ease of access of air to oxidation sites. The off-gas signature associated with the self-heating process appears to be quite complex.

INTRODUCTION

When coal is exposed to air an exothermic reaction takes place (Baum, 1981), which given the right conditions leads to a rise in temperature of the entire coal mass. Whether or not self-heating will occur is governed by the competing mechanisms of heat release from the oxidation reaction and heat losses by convection, conduction and radiation to the surroundings. If heat losses are minimal the coal will be under adiabatic conditions, which promotes self-heating.

One of the most significant hazards faced by longwall operations is coal self-heating leading to spontaneous combustion, which often results in millions of dollars in lost revenue and at frequent intervals has resulted in loss of life. A better understanding of the self-heating process and more definitive ways of assessing the risk of an incident will help improve the management of coal mining, resulting in the prevention of future major disasters.

Preliminary results from new studies of bulk coal self-heating using a 2-metre adiabatic column provide a much clearer picture of the importance of moisture transfer in the self-heating process and the off-gas signature associated with a heating.

BULK SELF-HEATING TESTS

Bulk sample self-heating tests of coal have been applied to a limited extent using medium-scale (40-1000 kilogram) test equipment. Stott (1980) reported the use of a 5m long, 0.6m diameter vertical container in the US. This experiment ran for five months and the coal temperature only rose to 45°C due to insufficient insulation of the outside of the column by normal means. It was recommended that a similar smaller apparatus be constructed with approximate dimensions of 2m long and 0.5m diameter. Chen (1991) followed these recommendations under the supervision of Stott, and built a so-called "Full-Scale Experiments Apparatus", which was 2m long and 0.3m in diameter. The equipment was used to study New Zealand coals ranging in rank from lignite to high volatile bituminous (Stott and Chen, 1992). However, a limitation of these tests was that they did not go beyond 120°C, and thus did not show the full extent of thermal runaway that occurs leading to self-ignition.

Li and Skinner (1986) used a 244cm long cylinder with a 61cm diameter, which featured a conical bottom. Their study showed the development of a hotspot in two tests performed on Black Thunder (Wyoming low sulphur, subbituminous) coal of 10% moisture and no hotspot development with 30% moisture. Monazam, Shadle and Shasmi (1998) were able to model these results using finite difference methods.

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Akgun and Arisoy (1994) also performed investigations using a column. The column was 3m long with a diameter of 0.3m. The features of this column included one spiral heater wound around the aluminium plate of the column’s inner wall. The column was mounted at an angle of 25° from the horizontal - because airflow in stockpiles is neither vertical nor horizontal (Akgun and Arisoy, 1994). However airflows in a stockpile were not approximated by the 200 litres/hour airflow at 75°C temperature, which was used. These airflows were used “in order to reduce the experimental time” (Akgun and Arisoy, 1994). Such airflow can never be referred back to the circumstances under which stockpiles are subjected in real conditions. Also, the use of one heater cannot provide adiabatic conditions along the length of the column.

Arief (1997) developed a column self-heating test apparatus at The University of Queensland (UQ) measuring 2m long and 0.2m in diameter (Figure 1), which was a modified version of the column used by Chen (1991). As suggested by Chen (1991), and from a practical viewpoint, the diameter of the column was reduced so that the amount of sample needed to undertake the experiments was relatively small and the column could be loaded and unloaded relatively easily. The column was used with as-received coal samples that would incorporate a range of particle sizes and moisture states, closely resembling natural conditions. The results of this preliminary use of a medium-scale apparatus to test Australian coals remain unpublished other than a brief conference proceeding paper (Arief and Gillies, 1995). A limited number of tests were performed on coals from two mines near Ipswich up to temperatures of 120°C, and hence the equipment has not been fully utilised for spontaneous combustion research.

AFIG. 1 - Schematic of UQ 2-metre column self-heating apparatus (modified from Arief, 1997)

Large-scale (16 tonne) self-heating tests have been successfully applied at the Queensland Safety in Mines Testing and Research Station (SIMTARS), with the limitation that it can take 6-12 months to generate one result (Cliff et al., 1998). The time taken for these tests is dependent on the physical parameters used in the test (e.g. coal size, moisture content etc.) and the rank of coal, which affects the reactivity. Smith, Miron and Lazzara (1991) performed large-scale spontaneous combustion studies using a 13 short ton chamber, with high-volatile C bituminous coals. One of these coals reached thermal runaway near the centre of the coalbed after 23 days from an initial starting temperature of 30°C.
In late 2001, the 2-metre column used by Arief (1997) was refurbished and recommissioned at UQ (Beamish et al., 2002). Since then, 17 test runs have been completed with a 100% success rate. From these tests it is clear that the column has a number of significant advantages. These are:

- The test is performed on as-received bulk coal (60 litres or 40-60kg of coal depending on packing density used) up to a maximum temperature of 250°C. This maximum temperature has been progressively established to maintain strict safety standards during testing. To achieve these temperatures, it was found necessary to remove the carpet inner originally used by Arief (1997). The supposed purpose of the carpet inner was to act as a baffler on the sides of the column to prevent air channelling. However, tests on coal before and after the carpet was removed showed no signs of significant changes to the self-heating development in the column.
- Tests results are available in days, with a full history of the self-heating development of the coal. The longest test run to date has taken 28 days to complete, with the majority of tests taking less than 14 days.
- Moisture effects are clearly visible.
- "True" off-gas is liberated and monitored from the self-heating process of the coal, including any moisture driven and gaseous feedback reactions leading to "fire-stink".
- Hotspot development and propagation can be quantified.
- Direct impacts of changes in airflow rate, particle size, air temperature and starting coal temperature can be assessed.
- Effects of the presence of pyrite and seamgas can be assessed.
- Simple coal quality relationships can be determined for any mine. These can also be related back to small-scale adiabatic $R_{70}$ testing.
- Mine strategies to control self-heating can be assessed, including:
  - ventilation changes
  - inertisation
  - inhibition

**COLUMN TEST PROCEDURE**

Beamish et al. (2002) provide a full description of the 2-metre column. A high-volatile bituminous coal sample was supplied from the mining face of a Bowen Basin mine. A size distribution was measured for the as-received sample prior to loading into the column, with a top size of 75mm being applied. The column has a load capacity of approximately 60 litres and for ease of loading 3 x 20 litre plastic sample buckets were used. The column was sealed at the bottom and coal loaded in through the top. As the coal level reached each thermocouple it was pushed into the centre of the coal. Once the column was full, the lid was fitted and sealed and the outlet hoses connected to a water trap, which was in turn connected to the outside atmosphere. At this stage the column was completely sealed with the outlet hose being blocked off and the outside wall heaters of the column were used to stabilise the coal temperature to 40°C in this particular test. Other starting temperatures can be used if the minesite wishes. This process usually happens overnight and the next morning the outlet was reconnected to the outside atmosphere and the air was turned on with a flow of 0.5 litres/minute. Exhaust gas emissions were monitored periodically with a Minigas, which measures oxygen, carbon monoxide, methane and hydrogen sulphide. Gasbag samples were also taken for GC analysis of other gases such as hydrogen, ethane and ethylene.

**MOISTURE TRANSFER DURING SELF-HEATING**

The temperature history of the coal self-heating is shown in Figures 2 and 3. Stages of heating development have been superimposed on the graphs. The corresponding temperature profile in the column is shown in Figure 4.
In the initial heating stage, the airstream transfers moisture from the lower part of the column to the upper part. This results in significant temperature increase due to the heat of wetting of the coal, which is exothermic. During this stage, one of the key off-gas parameters, Graham’s Ratio – defined as the ratio of the carbon monoxide concentration to oxygen deficiency expressed as a percentage, also increases as shown in Figure 2. This is most likely a combination of surface oxidation of the coal and oxygen access to the macropore system of the coal. This effect is even more pronounced in low rank coals.

After approximately two days, the coal becomes moisture saturated and evaporation begins to dominate the temperature history, with resultant cooling of the lower half of the column. The time to moisture saturation...
depends on the difference between the starting moisture of the coal and the moisture holding capacity of the coal. The Graham’s Ratio however, does not begin to drop in response to this cooling effect until a day later.

FIG. 4 - Temperature profile of a high-volatile bituminous coal self-heating in the UQ 2-metre column

Between day 4½ and 6½ a transition stage occurs, where the overall temperature and off-gas conditions remain reasonably static. This is followed by a significant increase in temperature between 37-118cm in the column due to coal oxidation, and a decrease in temperature between 118-145cm due to evaporation. During this stage the Graham’s Ratio remains fairly constant with minor fluctuations in response to diurnal changes, which also affects the coal temperature. This test was conducted in the middle of winter when the diurnal flux was at its greatest.

After day 11 there is an exponential rise in the Graham’s Ratio, which goes off-scale after day 14 due to limitations of the instrumentation. A gasbag analysis beyond this point indicated 4.49 and 6.62 for the Graham’s Ratio at days 15 and 16 respectively, consistent with the rampant self-heating taking place in the column. The significance of this dramatic increase in oxidation rate can be attributed to the coal having reached its critical moisture content. Hamilton (2001) showed that such a value exists for different coals by doing repeated R₀ tests at different moisture contents until thermal runaway could be achieved. For the subbituminous coals tested he found that this critical moisture value was below the air-dried moisture content of the coal and was also dependent on the mineral matter content of the coal.

By day 12 the hotspot is clearly visible at 73 cm from the air inlet (Figure 4), and by day 15 it begins to rapidly migrate towards the air inlet as the leading edge of the hotspot dries the coal and gets first access to the air.

OFF-GAS ANALYSIS OF GASBAG SAMPLES

The gas evolution associated with the column heating is shown in Figures 5 and 6. The coal has some residual methane (265ppm) and carbon dioxide (2.3%) present as seamgas, which desorbed from the coal while it equilibrated to 40°C. During the initial stage of moisture condensation there is a noticeable increase in hydrogen and carbon monoxide (Figures 5 and 6 respectively). This is followed by a steady decline in gas levels until the critical moisture content of the coal is reached at day 11 and a hotspot develops. At this point the carbon monoxide shows a 40% increase, while at the same time the hydrogen concentration increases by 60% and the carbon dioxide increases by 20%. The corresponding hotspot temperature at this stage was 70°C.

By day 13 the hotspot has reached a temperature of approximately 80°C. The hydrogen has increased by over 300%, carbon monoxide by over 400%, methane by 100% and carbon dioxide by 80%. Ethane and ethylene are absent and do not appear until much higher temperatures are reached.
IMPLICATIONS OF SELF-HEATING DEVELOPMENT FOR LONGWALL OPERATIONS

Moisture transfer is the key factor in self-heating development. If this does not take place then a heating cannot develop. Any form of air leakage path is capable of generating a heating. If it begins to dry the coal out below its critical moisture content then accelerated oxidation will take place. The stages of heating development in the column closely match reports from mines that have experienced heatings in the past. Visible signs of sweating would appear on the return side of a heating due to the moisture transfer of the airstream. The hotspot itself may be only very small and just inbye of the free surface of the air intake point, where temperature elevation of the coal would be virtually undetectable.
The accelerated heating due to “dried coal” oxidation has implications for gas drainage, where this is applied. Dry, dusty coal will grab any moisture available from the ventilation and begin to heat in response to the heat of wetting. This will in turn increase the rate of oxidation, which will in turn provide another heat source. In essence this would be double jeopardy. A situation such as this has already been simulated in the UQ 2-metre column (Phillips, 2002). Instead of a small hotspot developing the dimensions of the hotspot were grossly enlarged and the rate of self-heating was far more rapid than usual.

Interpretation of off-gas signatures requires careful monitoring and sound technical experience in understanding the stages of a heating. There are no simple rules of thumb that can be applied except that increasing gas trends warrant closer examination. With the assistance of bulk testing in the column patterns of gas evolution can be identified for individual coals that take into consideration the bulk chemistry that is taking place. This has been a major deficiency in past investigations and confusing information has been generated that may be providing a false sense of security to some mining operations.

**CONCLUSIONS**

As soon as coal is exposed to the air it will begin to dry. Moisture removed will be transferred further downstream by the air current and cause the coal temperature to rise due to the heat of wetting effect. Once the coal dries to a critical moisture level (below the air-dried moisture content), rapid oxidation takes place causing the coal to self-heat and generating a hotspot. This hotspot will then migrate towards the air source and if allowed to go undetected can propagate to an ignition once it daylight’s to a free surface.

Off-gas signatures are unique to individual coals during a heating and may vary dependent on the conditions to which the coal is subjected as well as the intrinsic properties of the coal. It is important therefore to monitor the changes occurring in gas evolution with time. Bulk coal self-heating tests provide the opportunity to recognise off-gas patterns that can assist mine operations to identify the development of an underground heating. In the example presented in this paper, the stages of heating development are reflected by the off-gas evolution. For this particular coal, hydrogen and carbon monoxide give the earliest indication of a heating. Closely monitoring the Graham’s Ratio would provide a definite indication of heating development.

As more tests are completed with the UQ 2-metre column, a far better understanding of coal self-heating is being obtained. Further study is needed on the effects of physical parameters such as airflow rate, particle size and, pile porosity. These will be conducted in parallel tests using two columns.

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**REFERENCES**


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